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STRUCTURAL DESIGN

VOLUME I

ELEMENTS OF STRUCTURAL DESIGN

BY

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PREFACE

It might be well to say here what is repeated in the text: that this work presupposes a knowledge of mechanics, stresses, and the mathematics on which they depend.

The experience of the author in teaching and in practice has led him to believe that no available book presents all structural subjects so concisely that they can be covered in the time usually allotted by technical schools. Moreover, the fundamental principles of shop practice and erection, which govern the designer at every step, are not clearly developed. These faults the author has attempted to correct, using only orthodox methods of presentation.

This volume considers wooden structures and the fundamental principles of design in steel. Should this work receive recognition, it is intended to follow it by a second volume on the "Design of Simple Structures," plate girders, viaducts, truss bridges, mill buildings, high office buildings, and standpipes, and a third on the "Design of Advanced Structures," cantilever, movable, and suspension bridges and arches. It has been thought best for the present to omit the data usually found in handbooks.

As a considerable portion of this treatise is original data, corrections and suggestions will be especially welcome. However, a great deal of pains has been taken to eliminate errors.

The writer wishes at this time to acknowledge his references. These include almost every American authority on the subject. Many of them have been referred to at the proper place in the book. Particular mention should be made of the *Engineering News*, *Engineering Record*, and Transactions of the American Society of Civil Engineers. The last two kindly allowed the

reproduction of their illustrations. The aid rendered by various manufacturers is acknowledged in the text.

The author desires to thank the following gentlemen for assistance in revising manuscript: Registrar Tarbell and Profs. McCullough, Jones, Lose, and H. S. Dornberger of the Carnegie Technical Schools and Messrs. Gordon, Allen, and R. S. Dornberger.

H. R. T.

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ELEMENTS OF STRUCTURAL DESIGN

CHAPTER I

MATERIALS

Art. 1. Introduction

THIS work is intended for students and draftsmen who understand analytical mechanics, mechanics of materials, and methods of determining stresses either by graphics or by computation. On this account, only a synopsis of results will be given in sample designs. However, the reader should check each operation to be sure that he thoroughly understands it.

The writer believes that the difficulty usually found in teaching structural work arises from the fact that fundamental principles are not clearly developed and illustrated by simple examples at the start. It is axiomatic among practical men that a thorough knowledge of details should precede any attempt at design. Yet this well established principle is often ignored in technical training.

We shall consider first the materials and their commercial shapes, which, often altered in manufacture, are used in structural work; next, the organization of companies to handle them; afterwards, their machines, their capacities, and the way in which they operate to transform the original shapes; then the means by which a member carrying a given stress can be most economically fabricated to carry its load; next the methods of fastening together these different members; and finally, the design of the finished structures.

In order to serve as a reference book for the student and draftsman, much data will be given which will be supplanted in practice by the slightly different standards of the locality or

of the company concerned. For example, the common sizes of timber as given in Art. 15, vary somewhat with the locality and date. They are inserted, however, to give an idea about how they run, to serve as a standard for student designs, and to be used where other information is not available. Data of this sort should not be memorized.

Art. 2. Growth and Characteristics of Timber *

The advantages of timber are that it is light, cheap, abundant, and easily worked or altered. On the other hand, it is accessible to vermin and insects, is combustible, quite perishable, and weak in shear and bearing perpendicular to the grain. These weaknesses are more fully discussed in Chapter III. The approaching exhaustion of our reserve supplies has partially neutralized its advantages of abundance and cheapness.

Timber is cut from the trees in a manner familiar to all. Those woods which are used in construction are almost entirely exogenous. These grow by formation of new wood each year on the outer surface. On a transverse section of the tree, they appear as rings whose number equals its age in years. Each ring is composed of two parts: an inner portion called spring wood which is soft and lacking in strength; and an outer part called summer wood which is hard and strong. Hence the more of the latter, the stronger the timber is. In the conifers (pine, spruce, and hemlock), the summer wood is dark and the spring wood light; in the broad leaved trees (oak, maple, birch), the reverse is true. When these annual rings are narrow, wood is said to be fine grained; if wide, coarse grained. When fibers are not parallel to axis, as in hemlock, it is called cross-grained; when wavy as in maple, wavy grained or curly. The figuring of bird's-eye maple is due to the concentric circles appearing in a tangential section of unevenly growing wood.

A zone of the wood next to the bark and 1 to 3 inches wide is lighter than the remainder and is termed sapwood because it is

* References for Timber: Bulletin No. 10, U. S. Forestry Div., Agricultural Department, 1895; Johnson's "Materials of Construction"; Kidder's "Building Construction and Superintendence," Part II. Snow's "Principal Species of Wood."

active in carrying the sap. The interior of the tree called heartwood is inert and fulfills only the mechanical function of helping to sustain the tree. Sapwood is weak and subject to rapid decay owing to the great amount of fermentable matter contained therein.

Outside of the tree is the bark, a rough scaly covering whose appearance is often of assistance in determining species. In preparing the tree for use as lumber, the bark should be sawn or stripped off, as it interferes with seasoning.

Inside of the bark, sap and heartwood, spring and summer wood alike, is made up of "wood fibers" and "medullary" or "pith" rays. The former are small hollow cells about 0.1 inch long and .001 inch in diameter. Their greatest length is ordinarily parallel to the axis of the tree. Pith rays are somewhat similar but smaller cells, extending radially. Much the greater part of the timber is composed of wood fibers and this gives rise to some of its structural peculiarities.

In the first place, as we should expect, its principal strength lies in a direction parallel to the axis of the tree. Other ways it is weak, as in shearing along the grain or in bearing perpendicular to the grain.

The shrinkage of timber is also affected by this peculiar composition. Roughly, green timber is one-half water; 2 per cent is a constituent part of the cells of the sapwood; 18 per cent saturates the walls of all cells; the remaining 30 per cent fills the cavities. All but a small part of this moisture may be expelled by drying. However, on exposure to the air, dried lumber takes up moisture until a 10 to 15 per cent content is reached. This is the standard for seasoned lumber.

There are three reasons for seasoning. First, to increase its strength (Art. 6). Next, the large amount of water and sap in green stuff affords a favorable condition for the growth of those germs which cause decay (Art. 3). Finally, the removal of moisture as it dries out, alters the dimensions of the timber. Let us study this change with care.

As one might expect, the relative change in the direction of the main axis of the cells is small; perpendicular to said axis, it is large. Radially, Fig. *2a*, it is held in a measure by the pith rays. Tangentially there is no restraint and more of the shrink-

age takes place in this direction. For this reason, green lumber of the shapes seen in Fig: 2*b*, takes, after seasoning, forms exaggerated in Fig. 2*c*. The longitudinal shrinkage of timber will average about one one-thousandth of the length. Approx-

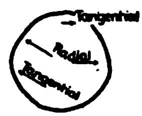


FIG. 2*a*.—Tangential and Radial.

FIG. 2*b*.—Original Shapes—Green.

FIG. 2*c*.—Resulting Forms—Seasoned.

imate values of radial and tangential shrinkage, deduced from U. S. Timber Tests, follow:

Variety.	Radial.	Tangential.
Soft pine, cedar, cypress, spruce	.020	.04
Hard pine	.025	.055
Ash, elm, poplar, walnut, maple	.030	.070
Chestnut	.040	.080
Oak	.060	.120

There are two methods of seasoning: (*a*) Air, (*b*) Kiln drying. In the former, the lumber is stacked in the open and, as far as possible, arranged to shed water yet give the air free access. This usually takes from two to three years and the lumber is never fit for some purposes. A kiln is a tight chamber through which a current of air of about 150 to 180° F. is passed. Hard woods should first be air-dried. Steaming is often employed to prevent checking or case hardening (see next article). Four to six days are required for 1 inch material and longer for larger stuff.

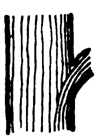


FIG. 2*d*.—Grain at a Knot.

At the junction of the limb and stem, fibers on the lower side run into trunk as seen in Fig. 2*d*. On upper side fibers are not continuous. For this reason a split made above does not run into the knot, while one made below does. When limbs die, they

break off, and are finally covered by the growth of the trunk, and give rise to the annoying dead or loose knots. Their weakening effect is fully explained by the structure of the timber at this point.

The color of the timber serves as a characteristic mark for many species, is a help in detecting decay, and adds to the appearance of certain finishing woods. Odor is of importance for the first two reasons. Cypress and hard pine, much alike in appearance, may be distinguished by the resinous odor of the latter. Resonance is that property which enables a substance to transmit sound. In timber, knots and other irregularities lessen this transmission. Decaying wood gives a dull heavy sound when struck with a hammer.

Values for the weight of various kinds of wood in pounds per cubic foot are as follows:

Variety.	Weight, green.	Weight, seasoned.
Hickory, oak	56 to 64	42 to 48
Ash, elm, cherry, maple, walnut	48 to 56	36 to 42
Hard pine, Oregon pine	40 to 48	30 to 36
Norway pine, cypress, hemlock	32 to 40	24 to 30
Cedar, spruce	24 to 40	18 to 30
White pine, poplar	24 to 32	18 to 24

Art. 3. Faults of Timber*

These may be divided into four groups:

(1) Defects of growth.
(2) Faults due to improper handling or seasoning.
(3) Bacterial decay.
(4) Injury caused by worms and insects.

Prominent among the first named are sapwood, shakes, and knots.

As already noted, sapwood is the weaker part of the tree. However, the rejection of pieces solely on this account is justified only in timber used for special purposes.

Shakes are of two kinds, the heart, Fig. 3*a*, and the cup,

* Year Book, 1911, Am. Soc. Test. Mat., pp. 166–172.

Fig. 3*b*. Only the larger trees are subject to them. Small shakes do no appreciable harm; those extending through the piece are serious and justify its rejection.

Knots are a grave source of weakness in timber. Whether in tension, flexure, or compression, the safe strength varies considerably with number and size of knots. A rough measure of their effect in weakening a piece is to consider them as open holes of equal size. In many cases they are objectionable for appearance's sake. It is generally impracticable to exclude knots altogether; however, they should be limited in amount.

(2) Faults due to improper seasoning and handling are wane, checking, and case hardening.

Where a portion of the exterior surface of the log appears on the sawn piece, as in Fig. 3*c*, it is called a wane. It signifies

FIG. 3*a*.
Heart Shake.

FIG. 3*b*.
Cup Shake.

FIG. 3*c*.
Wane.

FIG. 3*d*.
Checking.

the presence of sapwood, lessens the amount of timber, and may interfere with some uses.

Fig. 3*d* shows checking which is a separation of the wood fibers on the end of a stick, extending back into the piece but a short distance. It is caused by kiln drying and may be prevented by first seasoning in air for three to six months.

Case hardening is seasoning of the outer layers before the interior. The latter, as the operation goes on, shrinks and checks badly. It may be prevented by steaming, or, better still, by a preliminary air drying.

(3) There are three ways in which bacteria cause the decay of timber:

(*a*) Fungus growth.
(*b*) Dry rot.
(*c*) Wet rot.

Fungus growth may attach itself to the living tree, forming a knob on the exterior, while filaments from the same, in appearance much like roots, eat into the tree, and devour sap and fiber.

Another form attacks the sawn timber in much the same manner as the mould on a piece of bread. Moderately warm localities with moisture, but not immersion, and untreated lumber are fields suitable to its growth.

Dry rot is caused by bacteria which exist where the gases of decomposition cannot escape. Affected wood looks fairly firm, but readily crumples to powder when squeezed between the fingers. It spreads rapidly, contact not being necessary for infection. Moderate warmth, lack of ventilation, and moisture but not immersion, favor its growth.

A different kind of bacteria causes wet rot in timber. This variety thrives only where exposed to circulating air. The infected portion is generally wet, dark or dirty looking, and falls to pieces in the hand. It spreads by contact only. Moderate warmth and moisture without continuous immersion are favorable conditions, Wood is particularly susceptible to this form of decay when exposed to alternations of wet and dry.

To prevent decay due to these causes just outlined:

(*a'*) Season, thus in a measure depriving bacteria of their food.

(*b'*) Use preservatives. These are invariably disinfectants, that is, poison for the bacteria. See Art. 4.

(*c'*) Keep under fresh water. In tidal seas, there is danger from marine worms. (See below.)

(*d'*) Avoid conditions favorable to their growth as given above.

Of these, (*c'*) is the only method that is always dependable. Removal of wet rot already existing and observance of any one of above rules should prevent its spreading farther. For dry rot, in addition, all pieces in vicinity should be scraped and washed with acid.

(4) We shall limit ourselves to the consideration of marine worms.* Although there are insects which cause damage, they are largely limited to tropical climates.

The principal of these pests are:

Teredo or ship worm, about six inches long, and one-eighth inch in diameter.

* Colson's "Notes on Dock and Dock Construction." Eng. News, Vol. XL, p. 34.

Limnoria Terebrans, a small insect about one-sixth inch long resembling a wood louse.

Chelura Terebrans, in appearance like a sand shrimp. It is about one-quarter inch long and very destructive.

They inhabit salt water, attacking the timber between low tide and the bottom of the sea. Here they bore in the wood, in some cases leaving but a small percentage of the whole. For protection:

(*a*) Leave the bark on (partial).

(*b*) Use only certain kinds of woods, for example, palmetto, cypress, pine. . (Of doubtful value.)

(*c*) Drive flat-headed nails of iron, copper, or zinc close together. They should extend from a foot below the ground to high water.

(*d*) Cover timber with tarred paper and zinc or copper sheathing from a foot below the ground to high water.

(*e*) Creosote the timber. This is the best remedy and will be taken up in the next article.

Art. 4. Preservative Processes for Timber

The lack of durability in timber is caused by (1) rot, either wet or dry, (2) marine worms, and (3) fire. Means for preventing destruction by these agencies will be discussed in this article.

The life of timber if fully exposed to marine worms, is very short, say a year or so. For an untreated railroad tie it is six to twelve years, according to species of timber and location. The use of tie plates is expected to increase this. The life of untreated timber if exposed to the weather is ten to twenty years; if housed, forty years or more. Preservatives will considerably prolong durations given above.

If timber be kept under water it will last indefinitely. Well preserved pieces of wood have been found in the wet strata of bygone geologic ages.*

Timber to be buried in the ground should be charred or dipped in coal tar. In the latter case, seasoning is necessary; in the former it is advisable.

Creosoting, if well done, is the best of all preservative processes. The material, creosote, is obtained from the dead

* Eng. News, Vol. LIV, p. 555.

oil of coal tar by distillation. The idea of the treatment is to introduce it into the pores of the wood. As long as it remains there, the pests will not attack it. The creosote does not penetrate deeply, hence care must be taken in making fresh cuts in timber, as they may afford entrance to its enemies. The proper way is to cut and remove bark before creosoting. The creosote should not have more than $2\frac{1}{2}$ per cent of water and a specific gravity of not less than 1.04 at 100° F.

In the process, timber is first air dried for several months, then placed in large cylindrical vessels, subjected to steam at a pressure of 15 to 40 lbs. per square inch; next to a negative pressure of 12 lbs. per square inch; afterwards to creosote oil at a temperature of 120° F. and a pressure of 150 to 200 pounds per square inch. Five to twenty-five pounds per cubic foot of timber should be absorbed, several hours being required for the saturation. The larger amounts are for protection against marine worms. Creosote seems to make timber brittle and ill adapted to resist abrasive forces. The cost of the treatment is 1.0 to 1.5 cents per pound of creosote.

Various modifications of this process are in use but above is typical.

Burnettizing: In the same general manner, other chemicals may be introduced into the wood. In burnettizing, about $\frac{1}{2}$ pound chloride of zinc per cubic foot is injected at a cost of $2\frac{1}{2}$ to 6 cents per cubic foot. It bleaches out very rapidly with moisture or water, hence its field is very limited. Is likely to render timber brittle.

Kyanizing: Here the chemical is bichloride of mercury, but like the chloride of zinc, it rapidly dissolves out under the action of water.

Zinc-Creosote Process: For the sake of economy, the creosote and emulsion of chloride of zinc are simultaneously injected. Results, however, are not so favorable as for creosote alone.

Zinc Tannin, or Wellhouse Process: In this case, chloride of zinc is injected, followed by glue and tannin, these two latter substances forming an artificial leather which plugs up the pores in the outside so as to keep in the zinc chloride.

Other Processes: Many other process have been used among which we will mention:

(*a*) Creo-resinate—creosote, resin, formaldehyde.

(*b*) Water creosote—emulsion of creosote and water.

(*c*) Haselman—boiling in sulphates of iron, copper, etc.

"Fire Proof" Wood.* Wood impregnated on pressure with such salts as those of alum, ammonia, and the phosphates burns with much more difficulty than before treatment. It acts in two ways: first, a deposit is formed in the cells which retards the flame; second, a gas is given off that hinders combustion. This gas is sometimes quite offensive. There is no such thing as fireproof wood. It is simply fire retarding.

Art. 5. Varieties of Timber

White pine is an evergreen tree with a needle-like leaf. Timber is a light whitish color, does not warp or check, and is hence a good finishing lumber. Is easy to work but lacks strength. First-class white pine is scarce and expensive.

Hard, or long leaved Southern, or yellow pine timber is heavy, free from knots, has a reddish-brown tint and a resinous odor. Trees from which it is cut grow in the Southern States. It is very durable, very strong, very stiff, and stands well, but is hard to work. It is our best structural timber.

Norway pine is common along the Canadian border. The timber is a white wood with a reddish tint, and is soft and durable. Its strength and other characteristics are intermediate between those of white and yellow pine.

Oregon pine or Douglas fir is a western timber. It is much like yellow pine, except that wood is coarser grained. It is stiff, strong, and durable, and except for difficulty in working, an ideal construction timber.

There are three varieties of spruce; white, black, and red, all much alike. Timber is a light whitish or reddish color, soft, easy to work, of medium strength, warps and twists a good deal.

Hemlock grows in the northern United States and in Canada. Wood is light, of reddish gray color, lacking in strength, moderately durable, cross-grained, rough, and splintery. It shrinks and warps considerably in seasoning.

There are several varieties of cedar, all of which are light,

* Eng. News, Vol. LIV, p. 353.

soft, grayish brown or red woods. Timber is durable, seasons rapidly, shrinks and checks but little.

Several different species of cypress are found in the swamps of our Southern States. Wood is light, soft, easily worked, straight grained and free from knots. It warps and shrinks little and is used in finishing.

There are six varieties of ash of which the principal are white and black. Timber is heavy, tough, strong, and hard.

Three kinds of oak occur, white, red, and live. The latter may be distinguished by its very crooked limbs; the others, by the color of the timber. Oak is hard, tough, and strong. It is especially prominent among timbers by reason of its high shearing strength. It is difficult to work, shrinks and cracks badly in seasoning, but once seasoned, it stands well. Live oak is now very expensive and is used only for special purposes such as in wooden boats. Both white and red oak are extensively employed, but the former is more desirable in every way.

Beech is a white to light brownish timber, coarse textured, heavy, hard, and strong.

Chestnut is a light, soft, coarse-textured wood. Possesses only medium strength but is very durable.

Poplar or whitewood is a white or pale yellowish timber, very free from knots. Timber is light, soft, and weak, shrinks badly and warps considerably.

Maple makes a hard, tough, strong timber, white in color. It seasons and stands well.

Art. 6. Strength of Timber

Allowable values are in pounds per square inch, for good merchantable timber as received from the lumber yard. Formula for flat-ended columns:

$$S_c = a - b\frac{l}{d}$$

Where,

S_c = allowable compressive unit stress;

a, b = constants given below;

$\frac{l}{d}$ = greatest value of fraction $\frac{\text{unsupported length in inches}}{\text{least breadth in inches}}$.

BUILDINGS

Variety.	Tension.	Flexure.	Compression.		Bearing . . . to grain.		Shearing.	Modulus of Elasticity.
			a	*b*	Per.	Par.		
Chestnut	600	750	600	6	300	1000	60	1,200,000
White oak	1000	1000	1000	12	500	1500	150	1,600,000
Red oak	900	900	900	11	400	1200	140	1,500,000
Hemlock	400	600	400	4	200	750	60	900,000
Spruce	600	750	600	6	300	1000	80	1,200,000
White pine	600	750	600	7	200	750	60	1,100,000
Norway pine	800	900	800	9	300	900	70	1,300,000
Oregon pine	1100	1100	1100	14	300	1400	100	1,500,000
Yellow pine	1200	1500	1200	15	600	1500	100	1,600,000

TRESTLES AND BRIDGES

Variety.	Tension.	Flexure.	Compression.		Bearing . . to grain.		Shearing.	Modulus of Elasticity.
			a	*b*	Per.	Par.		
Chestnut	400	500	400	4	200	750	40	1,200,000
White oak	750	750	750	8	400	1000	100	1,600,000
Red oak	600	600	600	7	350	800	90	1,500,000
Hemlock	250	400	250	3	150	500	40	900,000
Spruce	400	500	400	4	200	750	50	1,200,000
White pine	400	500	400	5	150	500	40	1,100,000
Norway pine	500	600	500	6	200	600	50	1,300,000
Oregon pine	700	900	700	9	200	900	70	1,500,000
Yellow pine	800	1000	800	10	400	1000	70	1,600,000

Above values may vary considerably. Notice the marked weakness of timber in shear and bearing perpendicular to the grain.

The lower part of the tree is stronger than the upper. In a transverse section, the heart is stronger except in an old tree where it has begun to lose its vitality.

Tests have shown that boxing a tree for turpentine does not affect its strength. Time of felling makes no difference except as it may influence seasoning. Calling the strength of timber with 10 per cent moisture 100, we may say very roughly:

with 50 per cent moisture, its strength is 50; 40 per cent moisture, 55; 30 per cent, 65; 20 per cent, 80. Whether water is original or absorbed seems to make no difference. Resisting power does not seem to vary with the size except as the latter affects seasoning. Let us call the ultimate load for the usual accelerated test for a wooden piece 100: if the test lasts one day, the load becomes 75; one week, 65; one year, 60.

Art. 7. Uses of Timber

In all locations, durability and economy must be considered. In various situations, we have the following special requirements:

For posts, girders, joists, trusses, and roof timbers:

If stresses are light, ease of framing is the principal requisite; hence use spruce and hemlock.

For heavier stresses, strength is desirable and yellow pine, oak, or perhaps spruce or Norway pine may be employed.

For large timbers, either Oregon pine or yellow pine can be obtained in lengths up to sixty feet.

For under flooring, a cheap timber such as spruce or hemlock will do. A wood that will stand and wear well is demanded for the upper floors and thresholds; such are quarter sawn white oak, maple, and yellow pine.

For shingles use cedar, cypress, or white pine; for siding and clapboards, the last two may be employed; for doors, sash, blinds, inside and outside finish, use white pine or cypress.

Piles and cribbage may be designed of oak, elm, hard pine, cypress, spruce, and hemlock. For bridge ties, use hard pine and white oak.

Art. 8. Cast Iron*

Iron ores are dug from the earth, and placed in a blast furnace with coke or some other fuel and a flux like limestone to carry away the impurities. The resulting product is pig iron. This is heated in a cupola which is something like a small blast furnace. The melting is also similar except that charges

* Reference for irons and steels: Johnson's "Materials of Construction"; Campbell's "Manufacture and Properties of Iron and Steel"; Stoughton's "Metallurgy of Iron and Steel."

of coke and limestone are much smaller. The slag and the iron are tapped off, the latter into ladles which are poured as explained in Art. 16. After cooling, the box is taken apart, the projections on the casting cut off, and it is then placed in a tumbler to remove the sand. This is commercial cast iron. Let us examine it with especial reference to those imperfections which so limit it in structural work.

Cast iron consists of about 93 per cent of iron together with at least 1.5 per cent of carbon. The remaining portion is largely silicon, phosphorus, and manganese.

Carbon may occur in two forms, the combined and the graphitic. The former is the important element in cast iron, the effect of the other elements being, in general, to increase or decrease it and in that way influence the properties of the metal. A small amount makes a gray soft iron, easily worked and comparatively strong in tension. On the other hand, a large amount makes a hard brittle iron.

The effects of silicon and aluminum are similar, each tending to eliminate blowholes. A small amount of silicon diminishes combined carbon and hence softens the cast iron. A larger dose seems however to make it brittle.

Sulphur also makes iron hard and brittle and should not be allowed above .10 per cent. Phosphorus up to .70 per cent does not injure the metal, but helps it to fill the mould.

Manganese when alone seems to harden cast iron, but with much silicon present may soften it. Tends to counteract sulphur and silicon.

The strength of cast iron in tension varies from 10,000 to 40,000, with an average of 20,000 lbs. per square inch. In compression, tests on small, short pieces show an ultimate strength of 50,000 to 200,000. On full-sized columns, however, this falls to 20,000 to 40,000 lbs. per square inch. The reasons for this extraordinary drop will be discussed presently. The flexural strength will vary from 10,000 to 60,000 lbs. per square inch. An average value of the modulus of elasticity is 15,000,000 lbs. per square inch.

It is hard and resists fire and corrosion better than either wrought iron or steel. It cannot be hammered, bent, rolled, or forged. It is very likely to be brittle and it possesses little

elasticity or resistance to shock. It is liable to blowholes, to segregation,* and to stresses due to the shrinkage of the interior after the exterior has cooled. The last named are often termed "shrinkage" or "initial" stresses. Displacement of core in castings may occur and leads to the extremely objectionable eccentric sections.

As might be expected, such a material has proven unsatisfactory for engineering purposes. It is employed largely in locations where the stresses are small, compressive, and quiescent or nearly so; for example, in bearing blocks and washers. Cast steel is now displacing cast iron in many places.

Castings should be of tough gray iron with not over 0.10 per cent of sulphur. They must not contain any blowholes or other flaws. Test pieces 1 in. square must show a modulus of rupture not less than 40,000 lbs. per square inch.

Art. 9. Wrought Iron

Wrought iron may be defined as iron almost chemically pure, intermixed with more or less slag. A typical wrought iron will contain about 0.06 per cent carbon, 0.09 per cent silicon, 0.15 per cent manganese, 0.009 per cent sulphur, and 0.12 per cent phosphorus.

Pig iron from the blast furnace and iron ore are heated together in a puddling furnace. The resulting metal is squeezed and rolled out, giving it that fibrous quality which is characteristic of wrought iron.

The influence of carbon and silicon is to make the iron hard and brittle. There should not be over 0.25 per cent phosphorus, as it causes "cold shortness," that is, brittleness while cold. Sulphur should be limited to 0.05 per cent, as it is likely to cause "red shortness," or brittleness when hot.

The strength of wrought iron in tension along the grain varies from 45,000 to 55,000 lbs. per square inch, elastic limit from 23,000 to 40,000. Tensile strength crosswise of the grain will average 80 per cent of the above. Percentage of elongation, 5 to 30; reduction of area, 10 to 40 per cent. Shearing strength is, in either direction, 80 per cent of tensile.

* The concentration of certain elements in a part of the casting.

Due to the ductile nature of the material, short specimens do not fail in compression but grow stronger with increasing loads. Elastic limit is about the same in tension and compression. Longer specimens fail by buckling. Tetmajer gives:

$$S_c = 43{,}000 - 184s, \qquad s = 10 \text{ to } 112,$$
$$= 282{,}000{,}000/s^2 \qquad > 112,$$

where s is the slenderness ratio.*

Certain shapes, like rounds or squares, will not break in flexure owing to the ductility of the specimen. For some rolled sections or built beams, there is a chance for failure. If properly designed, they will show a modulus of rupture of 40,000 to 50,000 lbs. per square inch and a coefficient of elasticity of 25,000,000.

Wrought iron is a ductile metal; it can be welded, rolled, or forged; it will stand a great deal of abuse without injury. It probably resists corrosion better than steel. From above properties, it may be seen that it is an excellent structural material; however, the greater strength of steel has given it a preference over wrought iron. Its use since 1900 has been confined largely to blacksmith's work.

The following specifications for merchant iron, Grade "A," were proposed by Association for Testing Materials.

> Tensile strength, 50,000 lbs. per square inch or more.
> Yield point, 25,000 lbs. per square inch or more.
> Elongation in 8 ins., 25 per cent or more.

Must show a long clean silky fiber when nicked and broken. A piece shall bend cold 180 degrees flat on itself without fracture. Must be straight, smooth, free from cinder spots or flaws, buckles, blisters, or cracks.

Art. 10. Bessemer Steel

Melted pig iron is introduced into the converter and a blast of air is passed through it. After this has oxidized out the impurities, spiegeleisen, an iron rich in carbon and manganese,

* Maximum value of fraction $\dfrac{\text{unsupported length}}{\text{least radius of gyration}}$.

is added. The latter element unites with the oxygen, while the former gives to the steel the proper carbon content.

There are two methods of manufacture: the acid and the basic; in the latter, calcined lime is added to the molten steel to eliminate the phosphorus. While there is a slight preference for the basic on account of the lessened danger of an excess of phosphorus, engineers usually fail to specify either kind, but state permissible limit of this objectionable element.

The product may be divided into soft steel, containing 0.15 per cent carbon; medium, 0.30 per cent; hard, 0.50 per cent. The lower carbon content give us a ductile metal, possessing almost unlimited capacity for abuse. It is, however, weak compared with the hard steels which, on the other hand, are quite brittle. For structural work, we use either soft or medium, preferably the former if there is much forging. Where high stresses are to be resisted, medium steel is better. This paragraph applies also to open hearth steel as considered in next article.

Silicon increases strength and hardness and decreases ductility. Manganese in small quantities makes metal hard and malleable. Sulphur and phosphorus are both objectionable elements, making metal hot and cold short respectively. Both should be limited to very small amounts.

For reasons which will be given in the next article, the Bessemer process is used for structural steel only in inferior work. A great deal of the rail tonnage is Bessemer steel, but even here, it is being displaced by the open hearth process.

Strength and tests will be discussed under the head of open hearth steel, which it closely resembles. For rails, it is usual to specify the drop test and a chemical composition about as follows: carbon, 0.45 per cent; manganese, 0.90 per cent; silicon, not to exceed 0.20 per cent; phosphorus, not to exceed 0.10 per cent.

Art. 11. Open Hearth Steel

In this process, pig iron, scrap, and iron ore are subjected to an oxidizing flame in an open hearth furnace. When carbon has been lowered to the proper amount, spiegeleisen or ferromanganese is added which combines with the oxide of iron and

prevents further loss of carbon. As in Bessemer steel, we have the acid and the open hearth processes, also soft, medium, and hard steels. The effect of the elements is substantially the same in both cases.

Bessemer is the cheaper method but gives poorer material. Open hearth is more uniform and more reliable. The process is such that it can be better regulated to produce required composition of metal. Always specify open hearth for important structural work. We may use medium, soft, or rivet steel, the latter being very soft and ductile.

American Association for Testing Materials recommend following specifications:

No grade should have more than 0.08 per cent phosphorus, nor more than 0.06 per cent sulphur. The finished material should be free from injurious seams, flaws, or cracks. In addition the following requirements should be fulfilled:

	Rivet Steel.	Soft Steel.	Medium Steel.
Tensile strength, lbs. per sq.in.	50,000 to 60,000	52,000 to 62,000	60,000 to 70,000
Yield point, lbs. per sq.in. (not less than)	one-half tensile strength		
Elongation in eight inches (not less than)	26	25	22%
Cold Bend	180° flat on itself.	180° flat on itself.	180° around its own thickness.

The strength of steel will vary according to its impurities. Carbon is the most important element in its influence on strength. This in tension for a small specimen may be gauged by the physical requirements just given. Shear will average 80 per cent of the tensile stresses. Compression for small specimens has about the same elastic limit as in tension. For full-size pieces, we find a large reduction in breaking loads, particularly in compression. While data for definite conclusions are lacking, it looks as though the ultimate stress in a full-size column might, even with what are now considered sound details, fall as much as 50 per cent below that for a small-sized specimen. (Art. 69.)

The coefficient of elasticity is in the vicinity of 30,000,000 lbs. per square inch.

Many shop processes such as punching, shearing, and bending the metal, either hot or cold, cause stresses while still without load. These are called "initial stresses." Another cause is the rapid cooling of metal after heating for forging. To remove this undesirable condition, metal is heated to about 1200° F., and allowed to cool very slowly and uniformly. This process is called annealing.

Cast iron costs about 2.5 cents per pound when patterns are furnished; cast steel under the same circumstances, 4 cents. Steel or wrought iron will cost 1.2 cents as rolled or about 2.5 cents as fabricated, f.o.b. cars at shop.

Art. 12. Cast Steel and Alloys of Steel

The process of casting as outlined briefly in Art. 16, gives notable economy in the fabrication of shapes possessing an intricate form. But, as already pointed out, cast iron has faults which limit its application in structural work. Of late, the practice of making the castings of open hearth steel has grown steadily in favor. As in structural shapes, the material may be soft, medium, or hard steel. The latter, like rolled stuff, is unsuitable for structural purposes. The properties of the resulting castings are much the same as those of the metal from which it is poured. Thus, if soft steel be used, it will be ductile, show a large resistance to impact tests, have a high elastic limit and a definite yield point. Blowholes, cracks, and segregation are the faults to be guarded against. Complicated castings should be annealed.

We shall speak of but two alloys of steel, vanadium and nickel.* Both seem to increase strength to a remarkable degree without interfering with its toughness. In fact it is claimed that vanadium increases it.

Nickel steel has actually been used in bridge work, and it is doubtless the material of the future for long-span structures. The usual percentage is three to three and one-half and the steel to which it is added commonly open hearth. More car-

* See Waddell's paper, "Nickel Steel for Bridges," Trans. A.S.C.E., Vol. LXIII.

bon may be used than would be allowable without the nickel. Such an alloy will have an elastic limit about equal to the tensile strength of the steel and an ultimate strength 80 per cent greater. In compression, the excess will vary from 50 to 75 per cent, the latter for short struts. The coefficient of elasticity is unchanged. Nickel steel does not stand shop abuse as well as carbon steel, but it is nevertheless satisfactory. Shopwork such as punching, drilling, and chipping, will be more expensive. Nickel steel seems in general to resist corrosion better.

Art. 13. Paints *

Very little is known in regard to the theory of the preservation of wood by the use of paint. For steel, however, considerable has been done in this respect. It is now considered that rust, the principal enemy of steel, is due to the electrolytic action between the hydrogen of the water and the iron. Oxygen must be present. Also, some acids, for instance the carbonic acid always in the air, accelerate the rusting. To prevent this action, we have "inhibitors" or rust preventers. Alkalies act as such, also chromic acid and its salts. The former cannot be used with ordinary paints made of linseed oil, because they unite with the latter to form soap. This objection does not hold for the chromates, and they make excellent inhibitive paints.

Carbonic acid, oxygen, and moisture are always present in the air, and hence unprotected steel will rust. One excellent preventive method, encasing in concrete, will be taken up in Vol. II. We will now consider the other method, protection by paint.

This usually consists of an aggregate of pigment with a cementing material of linseed oil. Pure linseed oil is made by crushing flaxseed, and allowing it to stand and settle and thus purify. In this form, it is known as raw linseed oil, which dries or oxidizes very slowly. This process may be hastened by adulterating with japan drier or by boiling. Former makes oil poorer as a paint, while latter process is expensive.

* Cushman and Gardner's "Corrosion and Preservation of Iron and Steel." Ketchum's "Steel Mill Buildings," Chap. XXVII.

Pigment should be finely ground, preferably in oil. For it, the following substances may be used:

White lead (hydrated carbonate of lead), is employed for wood and for finished surfaces in steel. Disintegrates when attacked by corrosive gases and does not make a good bottom coat.

Red lead (lead tetroxide), is very stable, either on exposure to light or the weather. Is probably the best paint for metal. It is mildly inhibitive, but is improved by the addition of 3 per cent of zinc chromate.

Zinc oxide has a tendency to peel but when mixed with red lead makes a good paint for metal surfaces.

Iron oxide is sometimes used. It should be free from the hydrated oxide.

Carbon, when mixed with linseed oil, has a large covering capacity with a correspondingly reduced protection.

Structural work will average 150 to 250 sq.ft. per ton of metal. Common practice is to estimate ½ gallon per ton per coat.

CHAPTER II

COMMERCIAL SHAPES

Art. 14. Handbooks, Units, and Dimensions

THE leading manufacturers publish books which give the details and properties of the different shapes rolled by them, and a great deal of other data which are very useful to the draftsman and designer. Prominent among these are the handbooks prepared by the Cambria Steel Co., the Bethlehem Steel Co., and the Carnegie Steel Co. This additional information usually consists of safe loads for different shapes, either as a beam or as a column; radii of gyration and capacity of common types of built-up columns; values for rivets and pins; details of bolts, rivets, nuts, upset ends, eyebars, turnbuckles, sleeve-nuts, clevis nuts, pins, loop rods, and nails; weights and areas of plates and round or square bars.

A problem of frequent occurrence is to find the hypothenuse of a right-angled triangle when both legs are given in feet, inches, and fractions of an inch. This is a very cumbersome operation without the aid of tables of squares. Hall's Tables ($2.00) may be recommended, while Smoley's ($3.00) are still better. The latter also contains logarithms of numbers and sines, tangents, and secants, which may be used to advantage in figuring triangles. For large distances and for bridges built on a curve, a seven-place logarithmic table of numbers and trigonometrical functions is advisable.

A number of very good books have been written with the idea of still further assisting the draftsman. Such are Godfrey's "Tables," Osborn's "Moments of Inertia," and Sample's "Properties of Steel Sections."

In continental Europe, the metric system is employed in structural work. It would be convenient here, but the expense attendant upon such a change of units has hitherto prevented its adoption.

Dimensions of wires and thin plates are commonly given in gage numbers. For sheet steel, the United States Standard Gage is used and a sheet may be specified thus—1 Pl., 16″× No. 20 U. S. Standard gage×3′-4″. Unfortunately, there are several different "standards," no two of which are alike. This makes it necessary to name the one employed, as above, unless it is definitely understood by all concerned. A new gage, the "standard" decimal gage, has been recently adopted by the Association of American Steel Manufacturers, in which the gage is expressed in even decimals of inches. Its universal adoption would obviate the confusion now arising from the multiplicity of systems. Tables giving the equivalents of the different gages may be found in structural handbooks.

Except as above noted, the units of measurements are the foot, inch, and the thirty-second of an inch. Save the dimensions of pin-holes which will be taken up later, the following rules must be observed:

(1) All distances are to be computed to the nearest thirty-second of an inch.

(2) All distances except dimensions of castings, shapes, and plates, when 12″ or over must be expressed in feet and inches,—thus, 1′-6¼″, not 18¼″.

(3) The fractions of an inch must be reduced to its lowest terms, thus, 1′-6¼″, not 1′-6 8/32″.

(4) Machinists and pattern makers prefer dimensions up to two feet in inches. In structural drawings involving these classes of work, this rule may or may not be followed.

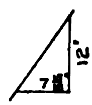


FIG. 14.—Method of Specifying Angles.

(5) The method of giving the dimensions of plates and shapes stated in the following articles must be used.

The only angle which the workman is supposed to understand is 90 degrees; all others must be given in bevels, that is, by drawing a right triangle with one side of the angle as the hypothenuse and the two legs drawn parallel and perpendicular to the other side. For example, an angle of 56 degrees and 30 minutes is expressed as shown in Fig. 14, the longer leg always being 12″.

Art. 15. Commercial Shapes of Wood

Fig. 15*a* shows the ordinary way of cutting up logs; the timber is then known as "bastard sawed." Fig. 15*b* shows the method taken to produce "quarter" or "rift" sawed lumber. The middle boards in Fig. 15*a* are sometimes sold for quarter sawed stuff. Bastard sawed is cheaper but does not stand well, on account of the tangential shrinkage.

With few exceptions, timber is sawn into rectangular shapes. Rough boards are commonly 1″, 1¼″, 1½″, 2″, or 2½″ thick. In the following table, a * indicates that the given size may usually be obtained, although the list will vary somewhat with the locality. Larger sizes may be had, but they are to be used with caution since they are more expensive per foot, B.M., more likely to season improperly, to contain sap-wood, decayed

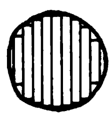


Fig. 15*a*.—Bastard Sawed.

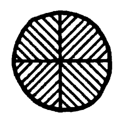


Fig. 15*b*.—Quarter Sawed.

heartwood, or other faults. Sizes in the lower left-hand corner are not common, since their use is inadvisable. Besides the reasons just given, a beam whose thickness is less than one-seventh the depth has a tendency to buckle. While good size timbers may be obtained up to a length of 60 feet, cost per foot for any given size increases rapidly above 20 feet. Stock lengths are usually 10, 12, 14, and 16 feet.

THICKNESS

Depth.	2″	3″	4″	6″	8″	10″	12″	14″	16″
2″	*								
3″	*	*							
4″	*	*	*						
6″	*	*	*	*					
8″	*	*	*	*	*				
10″	*	*	*	*	*	*			
12″	*	*	*	*	*	*	*		
14″	.....	*	*	*	*	*	*	*	
16″	.....	.....	*	*	*	*	*	*	*

When it is desired to give the wood a smooth finish, it is planed, $\frac{1}{16}''$ to $\frac{1}{4}''$ (varying with size) being taken off for each planing. Hence in specifying planed stuff, we should make it such dimensions that it could be readily cut from stock material; thus we specify flooring as $\frac{7}{8}''$, $1\frac{1}{8}''$, $1\frac{3}{8}''$, $1\frac{3}{4}''$, and so forth, to be made from $1''$, $1\frac{1}{4}''$, $1\frac{1}{2}''$, and $2''$ plank.

Figs. 15*c*, *d*, and *e* give forms for tongued and grooved timber. The purpose of the groove in the bottom of 15*d* is to lessen the effect of warping.

Shingles are wedged-shaped pieces of wood, $\frac{5}{16}''$ to $\frac{1}{2}''$ thick


FIG. 15*c*.


FIG. 15*d*.


FIG. 15*e*.

Tongued and Grooved Timber.

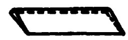


FIG. 15*f*.
Clapboard.


FIG. 15*g*.
Mouldings.

at the butt, 14 to 16″ long, and 3 to 14″ in width. A clapboard may be defined as a shingle 6″ long and 4′ wide.

A piece of wood, small in section and used for trimming is called a moulding, Fig. 15*g*. There is almost infinite variety to their shape, and they may often be obtained ready made from the lumber dealer, or ordered from the planing mill.

Art. 16. Commercial Shapes for Cast Iron and Steel Castings

Molten iron or steel is poured in a space formed by burying in sand a piece of wood called the pattern and then withdrawing it. This pattern is a duplicate of the desired rough piece except that it is a trifle larger to allow for the shrinkage of the hot metal. In order to make this space, the box which contains the sand should be cut by one or, for intricate castings, two or more "parting lines" or lines at which the box separates to remove the pattern. Holes are usually formed by "cores" which are prisms of a section same as the desired shape of the hole. These extend into recesses left by the pattern in the sand.

The principles which are of importance follow:

(1) The parting line must be so chosen that the pattern may be withdrawn. This is important since economy demands as few parting lines as possible.

(2) Surfaces which are shown on the drawing as parallel to the line of withdrawal of the pattern are tapered by the pattern maker about $\frac{1}{16}''$ per foot to prevent disturbance of the sand when the pattern is taken out.

(3) The thickness of the metal of the casting should be between $\frac{1}{2}''$ and $1\frac{1}{2}''$, preferably between $\frac{3}{4}$ and $1''$. If smaller than $\frac{1}{2}''$, it should not be used in important positions; if larger than $1\frac{1}{2}''$, make holes enough in it to cut down metal, taking care to conserve the necessary strength. Thus if it were required to make a casting $2'$–$0'' \times 1'$–$4''$ and $4''$ high, it should not be made solid but somewhat as shown in Fig. 16, the number of ribs being dependent on the strength required. This is done to avoid the initial stresses caused by the unequal cooling of different thicknesses. These alone are sometimes sufficient to break a casting. As noted in Art. 14, $1'$–$4''$ is often expressed as $16''$ for convenience of pattern maker.

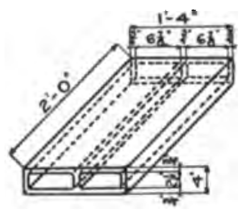


FIG. 16.—Typical Casting.

(4) Surfaces of revolution and plane surfaces are easiest to make and to handle, hence they should be used where possible.

(5) Two plane surfaces are not allowed to come to an intersection, but are joined by a curved surface of perhaps $\frac{1}{2}''$ radius, as the casting tends to crack at a sharp angle. The draftsman gives dimensions to the meeting point, but should show the rounding of the edges, without giving the radius, as the pattern maker takes care of this.

(6) Surfaces which must be exact and for which small variations of $\frac{1}{16}$ to $\frac{1}{8}''$ would not be permissible, should be marked " finish " or some abbreviation therefor. Designer should give dimensions of finished piece and the pattern maker will add the necessary amount. He also takes care of the shrinkage by using a shrink rule which is just enough longer than the ordinary rule to allow for the shrinkage of the metal when cooling, usually about $\frac{1}{8}''$ in a foot.

(7) Holes for bolts, pins, and so forth, may be marked,—" Core for ....dia. bolts," in case a rough fit is desired; or " Drill for .... dia. bolts," in case more exact work is wanted.

When designed for important duties, a bolt may be turned down and the hole made .002 to 003″ larger.

(8) Castings may be riveted but bolting is much more common.

Art. 17. Rolling

An ingot weighing twenty to thirty times as much per foot as the product and of sufficient length to furnish the desired amount of the finished shape, is passed between the rolls. These are so shaped that the piece is reduced to its proper dimensions by gradual steps. The hot metal tends to squeeze in between the rolls leaving projections called "fins," Fig. 17*b*, which tendency may be reduced by rolling with the joint at a different place.

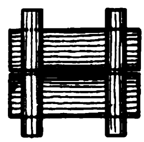


FIG. 17*a*.—Arrangement of Rolls in Universal Mill.

Auxiliary rolls, at right angles to the main ones, are often used to advantage. An example of this is the "universal mill," which rolls edges of plates as well as its flat sides, Fig. 17*a*.

Rolling iron or steel is a big trade in itself which we cannot enter into here. We will only attempt a few of the general principles as a means of understanding the common shapes and as a guide in case new sections are desired. The latter should be avoided except where the tonnage will be sufficient to justify it.

(1) Metal must not be too far from the axis of rolling;*

FIG. 17*b*.

FIG. 17*c*.

FIG. 17*d*.

FIG. 17*e*.

FIG. 17*f*.

FIG. 17*g*.

FIG. 17*h*.

for the consequent variation in the lineal speed of the rolls injures the metal.

(2) Projections at right angles to this axis must have a bevel. Thus a tee must be rolled as shown in Fig. 17*c*, and not as given in Fig. 17*d*.

(3) Reentrant angles can be made only by the use of auxiliary rolls after the final or "finishing" pass. Thus the shape shown

* The center of gravity line of the rolled shape.

in Fig. 17*e* must be rolled as seen in Fig. 17*f* and bent by the auxiliary rolls.

(4) Sections should be so designed as to cool evenly and thus avoid curling after rolling and the initial stresses due to one part cooling after another. A square rod must be rolled as shown in Fig. 17*g*, for, if first made square, it will shrink to the form seen in Fig. 17*h*.

(5) Plates ordered to a certain thickness may overrun their theoretic weight by from 3 to 10 per cent or even more. Actual amount for different sized plates may be taken from the handbooks.

(6) Unless a special price is paid, material is likely to vary from the specified length. The amount changes with the size, shape, and length, see Art. 58.

The limitations above given, especially (1), interfere with the development of an I-beam which possesses maximum economy as a beam. The Grey process of rolling, comparatively new, obviates this.

"The method of rolling comprises essentially a set of rolls with axes placed parallel to the web and working the inner profile of the beam, and a second set of rolls with axes normal to the web which works the outer faces of the flanges. The speeds of the two sets of rolls and their diameters are so related as to produce homogeneity of structure. A feature of commercial importance is the adjustable support of the rolls, permitting, for instance, a variation in weight by increase of flanges alone, or by increase of flanges and web in any specified proportion and that without changing rolls." *

Claims are made by those who own and control the patents that this process produces a superior metal, free from internal stresses. This is disputed and the counterclaim advanced that the metal is not as strong inch for inch as the old method of rolling. Tests which will settle the question of superiority should be awaited.

Rounds are often cold rolled. In this process, the hot rolled product is pickled in acid to remove the scale and rolled cold between chilled cast-iron rolls. Shafts made in this way may be obtained up to five inches in diameter. Rounds less

* Eng. News, Vol. XLVI, p. 387.

than one-quarter inch in diameter are wire drawn, that is, drawn cold through a groove smaller than the original diameter.

Steel which has a varying section or that which is too large to be rolled, must be cast or forged. The preceding article explains former process; in the latter the hot ingot is hammered into the required shape.

In the manner just indicated, many different shapes are rolled. For the present we shall limit ourselves to those used in structural work They may be classified as common, occasional, and rare, in accordance with the frequency of their occurrence in this treatise.

Common.	Occasional.	Rare.
Circular	T-beams	Deck beams
Rectangular	Z-bars	Bulb angles
Angles	Rails	Oblique angles
I-beams	Trough sections	Splice angles
Channels	Column sections	
....	Pipe	

The actual sections in which these shapes are commonly rolled may be obtained from the handbooks. The maximum lengths are also sometimes given there, or they may be secured by consulting the mills.

We shall now take up some of the principal facts in relation to each shape.

Art. 18. Circular Shapes

These, as their name implies, are true cylinders. They may be drawn, hot rolled, cold rolled, or forged. Circular shapes are termed wires if less than one-quarter inch in diameter; if more, rounds or rods. We seldom use less than one-half inch in structural work.

By " one inch rod," we mean that its diameter is one inch. In ordering or in specifying on plans, always describe thus:

$$8 \text{ Rounds, } 7\tfrac{3}{8}'' \text{ dia. } \times 1'\text{–}6\tfrac{1}{8}'';$$

or,

$$8 \bigcirc\text{s } 7\tfrac{3}{8}'' \text{ dia. } \times 1'\text{–}6\tfrac{1}{8}'',$$

always placing the length last.

Circular shapes may be obtained in all sizes from a fine wire up to a shaft two feet in diameter. Above 7″, rounds are forged; from ½ to 2″, they vary by sixteenths; from 2″ to 7″, by eighths; practice being slightly different for each company. There is no method of increasing area with the same grooves as in angles, Art. 20.

Wire is found in the cables of suspension bridges, while rounds are used in shafts, in rods for carrying tension, and for making bolts, rivets, pins, and rollers.

Art. 19. Rectangular Shapes

As their name indicates, these have a rectangular cross-section. They may be made by grooved rolls, by flat rolls, or by the universal mill as already explained. In the last two cases the increase in thickness is made by simply spreading the rolls.

A rectangular shape which has its width and thickness the same is called a "square"; if the larger dimension is greater than eight inches, it is termed a "plate"; if it is less, it is designated a "bar" or "flat," the latter term being usually applied to a thin section less than three-sixteenths of an inch in thickness.

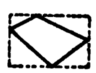


FIG. 19.
Irregular Plate.

Material is specified thus, 2 Pls., $36'' \times \tfrac{1}{2}'' \times 8'\text{–}4\tfrac{1}{4}''$, the first dimension being always in inches and parallel to the rolls. In this case, the 36″ dimension would have rolled edges,* the others being sheared. If above mentioned plate is irregular, as shown in Fig. 19, all edges are sheared edges. This is important since sheared edges do not make a nice fit or a neat appearance. Moreover, it is expensive to correct

* That is, it would if rolled in a universal mill. In "sheared plate" it is rolled somewhat larger and sheared to size.

by milling, Art. 37. As already noted in Art. 14, the thickness of thin plates or flats is often given in gages.

Common sizes are about as follows. Squares: from $\frac{3}{16}''$ to $\frac{9}{16}''$ by 32ds; from $\frac{9}{16}''$ to $2''$ by 16ths; from $2''$ to $3''$ by eighths; from $3''$ to $5''$ by quarters. Plates, bars, and flats usually vary by U. S. Standard gages where less than $\frac{1}{4}''$ thick; above that to $2\frac{1}{2}''$ in thickness by sixteenths. Above $15''$ in width, there is seldom a call for a plate more than one inch thick. Any width may be rolled in a universal mill, or may be sheared out. Common widths run from $2''$ to $5''$ by half inches; 5 to $12''$ by inches; 12 to $36''$ by even inches; 3 to 10 feet, by half feet.

These shapes in their different forms occur frequently in structural work: squares are used for ties; diagonals and chords carrying tension only are often made of bars; plates are employed in built-up girders, compression members, connection plates, and other places too numerous to mention.

Art. 20. Angles

The shape of the minimum thickness of an angle of a given length of legs is essentially that of two rectangles, joined together at right angles, Fig. 20*a*. The corners on the inside of the

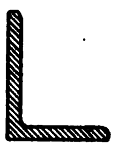


FIG. 20*a*.—Typical Angle.

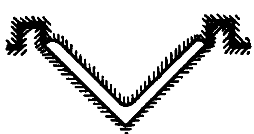


FIG. 20*b*.—Method of Rolling.

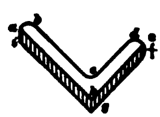


FIG. 20*c*.—Method of Increasing Section.

angle are eased by curves of a small radius, the values of which may be found in the handbooks. These curves must not be forgotten in detailing.

Fig. 20*b* shows the finishing rolls in position for the minimum thickness. Let the unshaded part of Fig. 20*c* represent a $4''\times3''\times\frac{5}{16}''$. Then $ah=4''$, $he=3''$, and the thickness of either leg is $\frac{5}{16}''$. Suppose it be required to roll a $4''\times3''\times\frac{3}{4}''$. We then raise the rolls an amount $ai=ef=sec\ 45°$ * $\times\frac{7}{16}''=\frac{5}{8}''$.

* Not always 45°, but near enough for illustrative purposes.

The distance $ig = ah$ and $gf = he$, the interior lengths and radii remaining unchanged. It will be noted, however, that the extreme length of each leg has been increased by $\frac{7}{16}''$ making the real size $4\frac{7}{16}'' \times 3\frac{7}{16}'' \times \frac{3}{4}''$, although it is spoken of as a $4'' \times 3'' \times \frac{3}{4}''$.

This theoretic shape is modified by the inaccuracies of workmanship, by the flowing out of the metal, or by a "finishing pass." The above reasons make the length of the leg of the angle somewhat uncertain; hence, in design and detail, we allow for a possible overrun. For the same reason, dimensions perpendicular to the edge called gages are always given from the sharp corner.

An angle is designated by the length of each leg and the common thickness. If one leg is longer than the other, it always comes first; if each is the same, both must be given, thus:

2Ls, $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}'' \times 24'\text{-}6\frac{1}{4}''$.
1L , $4'' \times 4''\ \times \frac{1}{2}'' \times 1'\ \text{-}0''$.

Angles are sometimes specified by weight, but both weight and thickness should never be given.
Thus,

1L, $4'' \times 4'' \times 12.8\# \times 1'\text{-}0''$.

Never,

1L, $4'' \times 4'' \times \frac{1}{2}'' \times 12.8\# \times 1'\text{-}0''$.

In the handbooks, angles are divided into equal and unequal legged, although the same general principles apply to each, and they are equally important. They may be obtained, increasing by small amounts, from a $\frac{3}{4}'' \times \frac{3}{4}'' \times \frac{1}{8}''$ to an $8'' \times 8'' \times 1\frac{1}{8}''$. Regular sizes vary only by sixteenths of an inch in thickness.

Angles are very common: alone or with 2 or 4 riveted together, they may be used for small tension or compression. They are employed for the flanges of girders and stringers. In built up columns and tension members, they fasten the plates together. These are but a few of their many applications.

Art. 21. I-Beams and Channels

If we neglect the curves which are used instead of sharp angles at the inside corners, an I-beam is made up of one large rectangle (the web), two smaller rectangles and four triangles (the flanges). The bevel of the sloping parts is 2″ in 12″ for standard sections. See Fig. 21*a*.

I-beams are rolled horizontally as shown in Fig. 21*b*. In order to increase the weight per foot, the rolls are simply spread farther apart, thus changing only the flange width and web

FIG. 21*a*.
Typical I-beam.

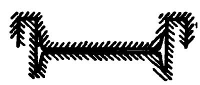


FIG. 21*b*.
Method of Rolling I-beam.

FIG. 21*c*.
Typical Channel.

thickness. The amount, w', of increase in pounds per lineal foot, divided by the height in inches times 3.4, equals the increased thickness in inches, either of the web or flange. That is,

$$t' = w'/3.4h.$$

They are specified by their depth and weight per foot, thus,

$$1\ I,\ 15'' \times 42\# \times 11'-4'';$$

the dimensions of the American standards, adopted January, 1896, are thereby understood. The sizes vary from a 3″×5.5# to a 24″×100#. They are used for beams, footings, and singly or latticed together as columns.

The Grey process, already explained in Art. 17, has made possible larger Is and those which contain more material in the flanges than the American standard. The former are theoretically far more economical either for a column or for a beam than the latter. If the new method will roll as good a quality of steel as the old, it will extend the new shapes into fields hitherto occupied by built-up sections.

Cut an I-beam in two along the web and we have a channel, Fig. *21c*. Methods of rolling and increasing the section are similar to those for I-beams.

Channels are specified by their depth and weight per foot, thus:

$$4\ [\text{s},\ 12'' \times 20.5\# \times 30'\text{-}0''.$$

Sizes vary from a $3'' \times 4.0\#$ to $15'' \times 55\#$. Shapes somewhat resembling channels are employed for small work as in expanded metal partitions. Where not otherwise stated, the dimensions of the American standard are understood. Channels are used for columns and for beams in places where an I-beam is not so convenient, for example, against a wall.

Art. 22. Occasional Shapes

T beams are composed of a flange, *abc*, decreasing in thickness from the center, and a stem, *bd*, increasing towards the top,

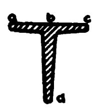


FIG. *22a*. Typical T beam.

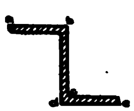


FIG. *22b*. Typical Zee bar.

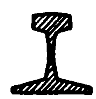


FIG. *22c*. Typical Rail.

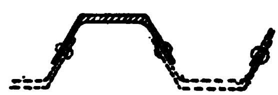


FIG. *22d*. Typical Trough Section.

Fig. *22a*. There is no method of enlarging by increasing the distance between rolls.

In case *ac* equals *bd*, the tee is called equal legged; if different, unequal legged. They are best designated thus:

$$2\ \text{Ts},\ 3'' \times 4'' \times 9.3\# \times 12'\text{-}6'',$$

the length of flange always being given before the depth of leg. Sizes vary from a $1'' \times 1'' \times 1.0\#$ to $4\frac{1}{2}'' \times 3\frac{1}{2}'' \times 15.9\#$. They are used principally in the roofs of buildings.

Zee bars, Fig. *22b*, are composed of three rectangles of the same thickness with some of the corners eased as shown. Methods of rolling and increasing size are similar to those for angles. They are specified thus,

$$4\ \text{Zs},\ 3\tfrac{5}{16}'' \times 5\tfrac{1}{16}'' \times 3\tfrac{5}{16}'' \times \tfrac{3}{8}'' \times 24'\text{-}2''.$$

First the flange, *ab*; then the web, *bd*; next the flange, *de*; and last, the common thickness. It will be noted that the increase in the length of the legs is shown, differing in this regard from angles. Since the two flanges are usually the same, one of them is sometimes omitted, thus,

$$4 \text{ Zs, } 5\tfrac{1}{16}'' \times 3\tfrac{5}{16}'' \times \tfrac{3}{8}'' \times 24'\text{-}2''.$$

Sizes vary from a $3'' \times 2\frac{11}{16}'' \times \frac{1}{4}$ to a $6\frac{1}{8}'' \times 3\frac{5}{8}'' \times \frac{7}{8}''$. Their principal use is in columns.

Rails, Fig. 22*c*, of many different kinds are rolled but the standards of the American Society of Civil Engineers are usually specified, as they may be more readily obtained. Sizes vary from an $1\frac{1}{2}''$ rail at 8# per yard to a 6″ at 150#. For the Society standards, the inclination of the top of the base and the bottom of the head is 13 degrees with the horizontal. It is not customary to spread the rolls to increase the weight.

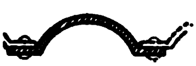


FIG. 22*e*. Typical Trough Section.

FIG. 22*f*. Portion of Phoenix Column.

FIG. 22*g*. Special Column.

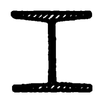


FIG. 22*h*. Typical H section.

Rails are specified by their depth, weight per yard, and name of standard, thus,

$$10 \text{ Rails, } 5'' \times 80\# \text{ per yard, A.S.C.E.,} \times 30'\text{-}0''.$$

The depth is sometimes omitted. Rails are used for railroad tracks, crane tracks, and in bearings and spread footings.

For the purpose of making a solid steel floor with sufficient strength to carry a load for a span of several feet, special shapes are rolled, so designed that they may be readily fastened together. See Figs. 22*d* and *e*. Angles and plates riveted to form trough sections, are now the standard construction for this purpose, see Vol. II.

Special column sections are rolled, the idea being to obtain a strong post with a minimum of riveting. The old Phoenix

column consisted of four or eight shapes fastened together as shown in Fig. 22*f*. It is now obsolete on account of the difficulty of inspecting, repainting, and making connections thereto. Jones and Laughlin roll a patented shape, a bent I-beam, which when riveted to a similar shape, give us a post as shown in Fig. 22*g*. The recently introduced H-shape, Fig. 22*h*, has become very popular as a column. (Art. 56.)

Pipes are sometimes used in structural work, but the method of manufacturing them does not belong here. Their actual diameters differ somewhat from the nominal, and tables in handbooks should be consulted in case their exact dimensions are required.

Art. 23. Rare Shapes

Among such may be mentioned the deck or bulb beam, Fig. 23*a*, which is simply an I-beam rounded off on the bottom. Manner of rolling, of increasing weight, and of specifying, are

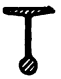


FIG. 23*a*.
Typical Bulb Beam.

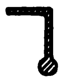


FIG. 23*b*.
Typical Bulb Angle.

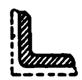


FIG. 23*c*.
Typical Splice Angle.

same as for I-beams. They are employed in place of the latter on ships.

In this place bulb angles are also used. These are angles with a swelled and rounded end, as shown in Fig. 23*b*. Manner of rolling is same as for angles.

Angles with the legs at more or less than ninety degrees may be had, but it is generally cheaper to bend an angle or plate to the required shape.

Splice angles, Fig. 23*c*, which are angles with the outside corner rounded so as to fit the inside of an angle, may be obtained; but it is preferable to plane off the corner and shear the edges of an ordinary angle.

CHAPTER III

WOODEN STRUCTURES *

Art. 24. Principles of Design

(1) Timber as ordinarily received is likely to be a little less than nominal size.

(2) The dimensions of timber perpendicular to the grain will vary with the amount of moisture it contains. If green timber be placed in a warm dry building, it will shrink; seasoned stuff out-of-doors will absorb moisture and swell. In a direction parallel to the grain, there is little alteration. (Art. 2.) Hence, where settlement is to be avoided, put grain of timber in line with the loads. If some pieces must be placed otherwise, make them as thin as possible. For example, the cracks in the plastering of dwelling houses are largely due to unequal settlement. This is caused by resting the studs on top of the joists, Fig. 24*a*, instead of passing between them as in Fig. 24*b*.

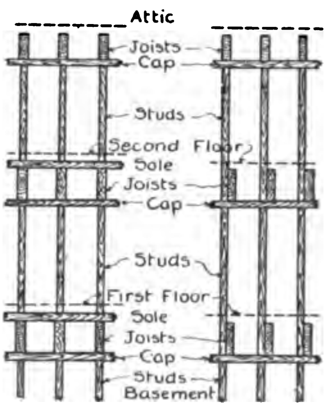


FIG. 24*a*. Poor Design. FIG. 24*b*. Good Design.

Methods of Supporting Floors in Buildings.

(3) When practicable, timber should not be placed where it will be subjected to alternations of wet and dry. This is in order to prevent wet rot, Art. 3. The expense of roofing wooden bridges has been considered advisable in many instances in order to prolong the life of the structure.

* "Structural Details," by H. S. Jacoby; "Building Construction and Superintendence," Part II, by F. E. Kidder; "Architects' and Builders' Pocket Book," by F. E. Kidder; "Roof Trusses in Wood and Steel," by M. A. Howe.

(4) Keep timber well ventilated to prevent dry rot. (Art. 3.) If it be necessary to build a member of two or more pieces side by side, they should be held an inch or so apart by blocks or washers. A hole is sometimes bored lengthwise through posts with crossholes top and bottom, partly for the same reason. The bricking up of the ends of joists or trusses, while common, is open to this objection.

(5) Where it is desirable to keep out the rain, arrange joints so that water would be compelled to run up hill before entering the structure. Thus, in the cornice shown in Fig. 24*c*, instead of details in 24*d*, we may employ arrangement of 24*e*. However, the rain which drove against the upright board would still tend to follow along the bottom of the horizontal board and into

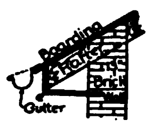


FIG. 24*c*.
Cornice.

FIG. 24*d*.
Incorrect.

FIG. 24*e*.
Incorrect.

FIG. 24*f*.
Good.

FIG. 24*g*.
Good.

Details of Cornice.

the wall. To prevent this, the cornice is designed as shown in Fig. 24*f*. Another method is to cut a small groove on the under side near the outer corner as in Fig. 24 *g*.

(6) The exclusion of birds and vermin must be borne in mind especially in residences and office buildings. Rat-proofing in San Francisco is now compulsory. Sparrows' nests in covered Howe trusses are a frequent cause of fire.

(7) A structure properly designed at all points for the usual loads is reasonably secure against hurricanes or earthquakes. In localities where these are especially prevalent, higher values should be assumed for the wind load and extra pains should be taken to fasten together the different parts.

(8) To protect a wooden structure from fire:

(*a*) Avoid small sections. A 10″×10″ wooden post will

stand a fire longer than an unprotected steel beam of equal capacity.

(*b*) Keep surfaces as unbroken as possible. A roof made of 3″ plank on purlins is a much better risk than one of 1″ boards on 2″×8″ rafters.

(*c*) Avoid enclosed spaces, for the fire there is difficult to reach.

(*d*) These enclosed spaces, if extending from floor to floor, are even more objectionable as they help the spread of flames.

(9) The strength of the wood in bearing perpendicular to the grain is much smaller than its compressive strength. A column is frequently designed as shown in Fig. 24*h*. However,

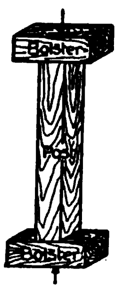


FIG. 24*h*.
Post with Bolster.

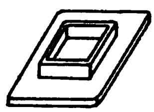


FIG. 24*i*.
Cast-iron Base.

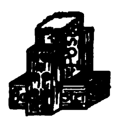


FIG. 24*j*.
Splice at Base.

except for very long columns, the bolster is much the weaker part. This may be remedied by making the latter of strong material, white oak for example. Also a casting as shown in Fig. 24*i* may be used or bolster and column spliced as in Fig. 24*j*.

(10) Lack of strength in shearing along the grain is perhaps its most important characteristic, weakening it much as a structural material. In splicing or joining a tension member, this fault makes it possible to secure but a fraction of the original strength. For the same reason notching a beam either top or bottom, produces, especially when near the ends, a large loss in capacity.

Art. 25. Accessories of Other Material

Steel, wrought iron, and cast iron are valuable aids in the design of wooden structures. In this article, when not otherwise mentioned, either of the first two may be used.

(1) Tension Members. For the reason given in (10) of preceding article and also that in Art. 27, iron finds frequent application in the transmission of tension. Fig. 25*a* shows how a rod may be used for this purpose. The bearing area at the notch should have a strength equal to that of the rod. If in addition, the latter be upset, Art. 43, its full strength will be conserved. In upsetting, the blacksmith enlarges each end of the rod until the excess is sufficient to provide for the cutting of the thread and the weakening due to forging. Dimensions

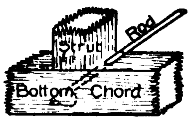


Fig. 25*a*.—Use of Rod as Tension Member.

Fig. 25*b*. Upset End.

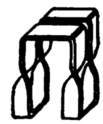


Fig. 25*c*, Fig. 25*d*.—Hangers.

of upsets can be obtained from handbooks. It does not pay if the pieces be short, small, or few in number. Fig. 25*b* shows an upset with a thread turned thereon. Eyebars, Art. 43, or loop rods, Art. 42, may also serve as tension members. In either case, pins, large special bolts, Art. 46, are used at the joints.

(2) Connection Plates. These are usually plates or bars, either bent or straight. Some will be taken up in later articles in this chapter. Of the many useful applications, we will call attention to the hanger, which may be found where one beam frames into another at right angles. Fig. 25*c* shows the single type; 25*d*, the double. The twist should be so made that the upper parts of the hangers lie flat against connecting beam. The size of the bar varies from $2'' \times \frac{1}{4}''$ to $4'' \times \frac{5}{8}''$. The strap bolt, Fig. 26*u*, is another excellent form.

(3) Reinforcement for Beams. For example, Fig. 25*e* shows the manner in which the steel is placed in a "flitched"

beam. Plate should be $\frac{3}{8}$ to $\frac{5}{8}''$ thick; the total width of the timber, about twelve times that of the plate; the plate, $\frac{1}{2}''$ less depth than the wood on account of overrun of steel and underrun and shrinkage of timber. An alternate spacing of $18''$ suffices for the $\frac{3}{4}''$ bolts.

FIG. 25e.—Flitched Beam.

(4) Bearing Blocks. These are usually of cast iron. Their function is to distribute the pressure over a larger area of masonry or timber. The latter case has already been discussed in (9) of the preceding article. Fig. 24*i* in that paragraph represents one type of a bearing block. A style known as the angle block is shown in Fig. 26*v*. Still another angle block is seen in Fig. 31*d*.

FIG. 25*f*. FIG. 25*g*. FIG. 25*h*. FIG. 25*i*. FIG. 25*j*.
Washers.

(5) Washers. The ratio of the bearing strength of timber perpendicular to grain to that of iron is very roughly one-twenty-fifth. If we allow for the screw thread, we find that about fifteen times gross area of rod is required in bearing. As this is an extreme case, standards are often a little less. For bolts where they are not in tension, full development is not necessary. Here the plate washer, Fig. 25*f*, or square washer, Fig. 25*g*, suffices. Either may be cast with a cored hole or made of rolled stuff with a punched hole. At any rate it should be $\frac{1}{8}''$ greater than threaded end. The diameter of the washer is two or three times that of the bolt. The thickness must not be less than $\frac{1}{4}''$. The remaining washers are cast. Fig. 25*j* shows an ogee washer, a very common form. Its thickness equals diameter of bolt equals one-half diameter at top equals $\frac{1}{4}$ diameter at bottom. Fig. 25*h* represents a lighter form, while Fig. 25*i* gives a washer where the rod enters at an angle.

(6) Keys. These are simple prisms of a uniform rectangular

section, usually of cast iron. However, they may be made of some wood which, like oak, possesses a large resistance to shear. Fig. 26*k* exemplifies their use.

(7) Fastenings.

(*a*) Nails, either cut or wire, are familiar to all. They are specified as twopenny (2d); threepenny (3d); etc., sizes varying from 2d to 60d. Dimensions are given in handbooks.

(*b*) Bolts. These are rounds with one end headed up and a thread turned on the other. The head is a prism either square as in Fig. 25*k*, or hexagonal. On the threaded end is placed a nut which also may be either square or hexagonal. Bolts are specified by diameter of cylindrical part and length *l*, Fig. 25*k*, thus, 240 bolts, $\frac{3}{4}'' \times 6''$ u.h., (under head) with square heads and nuts. Usual sizes of bolts are, $\frac{3}{4}$, $\frac{7}{8}$, 1, $1\frac{1}{4}$, $1\frac{1}{2}''$ diameters. Up to 24″ in length, they are carried in stock. Holes in wood are usually made the same size as the bolt.

FIG. 25*k*. Bolt.

FIG. 25*l*. Lag Screw.

FIG. 25*m*. Wood Screw.

FIG. 25*n*. Drift Bolt.

(*c*) Lag screws are bolts with the screw end pointed, but without nuts (Fig. 25*l*). They vary in size from $\frac{1}{4} \times 1\frac{1}{2}''$ to $1 \times 12''$ u.h.

(*d*) Wood screws, Fig. 25*m*, are somewhat similar except that the head is differently shaped and has a slot for driving. They are made in all sizes up to 6″ in length.

(*e*) Drift bolts, Fig. 25*n*, look very much like large spikes and are driven in a similar way into a bored hole. The diameter of the round hole should be 30 per cent less for the round bolt and 15 per cent less for the square. Ragging, that is, roughening the bolt, lessens its holding power.

(*f*) Dowels or dowel pins are double-ended drift bolts.

Art. 26. Joints

While almost any joint may be made entirely of timber, it will be generally possible to conserve a large percentage of the original strength only by the use of steel.

Where the members are fastened together by overlapping and bolting as in Fig. 26*a*, it is called a scarf joint.

A fish joint is one where the members abut and are fastened together at the side by timbers or plates called fish plates.

Let us take now the very simple fish joint shown in Fig. 26*b*.

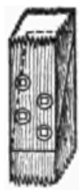


FIG. 26*a*.—Scarf Joint.

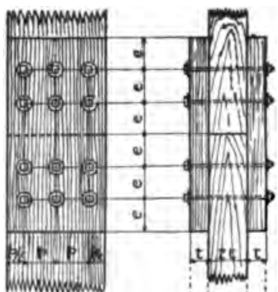


FIG. 26*b*. Fish Joint.

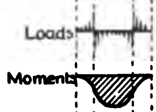


FIG. 26*c*.

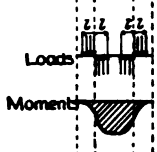


FIG. 26*d*.

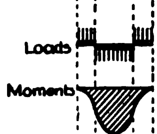


FIG. 26*e*.

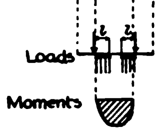


FIG. 26*f*.

S_s, S_s' = shearing unit stresses of timber and metal respectively.
S_t, S_t' = tensile unit stresses of timber and metal respectively.
S_f, S_f' = flexural unit stresses of timber and metal respectively.
S_{bo}, S_{bo}' = bearing unit stresses of metal against timber and metal respectively.

The true distribution of loads on the bolt with the diagram of bending moments is given in Fig. 26*c*. Fig. 26*d* represents our assumption; Fig. 26*e*, that usually employed. The latter arrangement gives much higher bending stresses than either of the other two. To neutralize this, it is common to employ high flexural stresses in connection with such assumption. Let us now investigate the safe capacity of such a bolt, using Fig. 26*d*. Here the bolt of diameter d will be loaded with the safe bearing pressure for such a distance l that the safe shearing or flexural strength is equaled. This distance l must not exceed t in Fig. 26*b*. By equating shear and moment to the resistance of the section of the bolt we obtain:

For shear

$$l = \pi d S_s'/4S_{bg};$$

For moment

$$l = d(\pi S_f'/32S_{bg})^{\frac{1}{2}}.$$

Fig. 26*f* shows the forces where the steel fish plates are used. Here above equations become,

$$l = \pi d S_s'/4S_{bg},$$
$$l = d(\pi S_f'/16S_{bg})^{\frac{1}{2}}.$$

By these equations the safe lateral resistance in pounds may be computed for a one-inch round bolt, drift bolt, spike, nail, wood screw, or lag screw. Other sizes will carry a load in proportion to the diameter squared. For double shear, double values. We have also added safe unit resistance to withdrawal. For screws this is based upon the area of the circumscribing cylinder. In nails and screws, area of the point is not considered. The quantities given are for buildings, for bridges use two-thirds of same. Amounts below are based upon proper design of e, p, and t, Fig. 26*b*.

This distance e should be such that the safe shearing stress is not exceeded. Assume this to be carried in the same depth as the bearing,

$$S_{bg} dl = S_s 2el$$

or

$$e = dS_{bg}/2S_s,$$

a mean value of which is 6*d* for pressure parallel to grain.

$S_f' = 12{,}000$ LBS. PER SQ.IN. FOR WROUGHT IRON AND 16,000 FOR STEEL

	Lateral Strength in Lbs. for Bolt 1" Dia. in Single Shear.								Withdrawal in Lbs. per Sq.in.					
	Bearing ⊥ to Grain.				Bearing ‖ to Grain.				Cut Nails.		Drift Bolts. Wire Nails.		Wood and Lag Screws.	
	Metal Plate.		Wooden Plate.		Metal Plate.		Wooden Plate.							
	Steel Bolt.	Iron Bolt.	Steel Bolt.	Iron Bolt.	Steel Bolt.	Iron Bolt.	Steel Bolt.	Iron Bolt.	⊥ to grain.	‖ to grain.	⊥ to grain.	‖ to grain.	⊥ to grain.	‖ to grain.
Chestnut	1000	840	700	600	1900	1600	1300	1100	150	120	120	100	300	250
White oak	1200	1050	880	750	2100	1800	1500	1300	250	200	200	150	500	400
Red oak	1100	1000	800	700	2000	1700	1400	1200	200	150	150	120	400	300
Hemlock	800	680	540	480	1500	1300	1000	900	150	120	120	100	300	250
Spruce	1000	840	700	600	1900	1600	1300	1100	120	100	100	80	250	200
White pine	800	680	540	480	1500	1300	1000	900	100	50	40	30	200	100
Norway pine	1000	840	700	600	1600	1400	1200	1000	120	80	50	40	250	150
Oregon pine	1000	840	700	600	2000	1700	1400	1250	150	120	70	50	300	250
Yellow pine	1400	1200	1000	850	2100	1800	1500	1300	150	120	70	50	300	250

On account of washer and nuts, p cannot be much less than $3d$, Art. 25 (5). As in riveted joints, the idea is to make strength of bolts equal to that of the net section. Economy of material for a tension joint may be promoted by arranging it as shown in Fig. 26g.

Above analysis gives uniform weight of bolts, no matter what diameter may be taken. Hence use as large a size as is

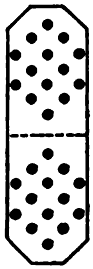


FIG. 26g.—Efficient Joint.

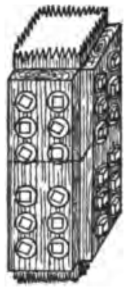


FIG. 26h.—Fish.

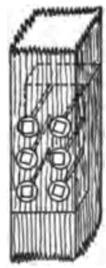


FIG. 26i.—Scarf.

Joints for Compression Members.

convenient and will afford a satisfactory distribution of the stresses.

We will now consider the following cases:

(1) Splicing a compression member.
(2) Splicing a tension member.
(3) Splicing a beam.
(4) A tension member entering a piece.
(5) A compression member entering a piece.
(6) A beam framing into another beam.

(1) Compression members may be spliced by:

(a) Fish plates on all four sides, Fig. 26h. Each plate should have not less than two rows of not less than two bolts each on each side of joint.

(b) Scarf joint which should be parallel and perpendicular to the compression as shown in Fig. 26i.

(c) Combination of the two.

Abutting surfaces are supposed to carry the load and the bolts are put in to render the member continuous under com-

pression. Properly built joints have an efficiency close to 100 per cent.

(2) Tension members are spliced by:

(*a*) Plain joint with 2 or 4 fish plates, either of timber or steel. Fig. 26*b* would do for small tensile stresses.

(*b*) Scarf joint, Fig. 26*j*.

(*c*) Combination of fish and scarf, often with keys, Fig. 26*k*.

Fig. 26*j*.—Scarf Joint for Tension.

(*d*) Fish plates if of steel may be bent and, if of timber, may be notched into the tie. Fig. 26*l* shows the latter case.

The author prefers (*a*). It should be remembered that the notching and fitting of complicated joints are expensive. Some

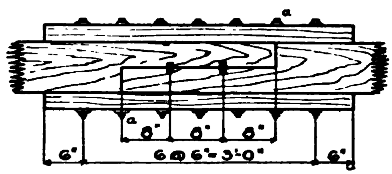


Fig. 26*k*.—Combination Joint for Tension.

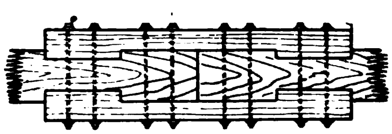


Fig. 26*l*.—Notched Fish Plate Joint for Tension.

of these may show higher efficiency in an analysis based on all elements working in unison. This, however, is an ideal which is not reached in practice. For example, in Fig. 26*k*, it is quite difficult to notch out so exactly that the load will be divided among the keys and bolts in proportion to their strength.

To illustrate the computation of a joint in tension, let us

determine the capacity of splice shown in Fig. 26*k*. Timber, white oak; cast iron keys; and 1″ steel bolts. Piece is 8″×8″, hence will have two rows of bolts. Using stresses for buildings, these two rows of five bolts each will carry,

$$10 \times 2 \times 1500 = 30000 \text{ lbs.}$$

The maximum stress on the net section of the $2\frac{1}{2} \times 8''$-fish plate occurs at *a*, and is,

$$15{,}000/(2\tfrac{1}{2} \times 6) = 1000 \text{ lbs. per sq.in. O.K.}$$

Taking the keys as 1″ in width, the capacity of each is,

$$8 \times 7 \times 150 = 8400 \text{ lbs.}$$

Half depth of key must be such that allowable bearing is not exceeded.

$$8400/8 \times 1500 = 0.70 \text{ in.}$$

We will make them $1\frac{1}{2}''$ deep. The total capacity of the joint is then,

$$30000 + 16800 = 46800 \text{ lbs.}$$

To obtain this capacity, fish plates must be extended to include four more rows of bolts to the right of the upper *a* and to the left of the lower one.

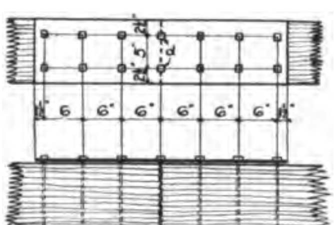


FIG. 26*m*.—Joint in Beam with Steel Fish Plates.

(3) Splicing for bending is to be avoided where possible. If necessary, use methods given in (2). As before, (*a*) is preferable. The plates may be planes parallel to the moment or in planes perpendicular thereto. In the former case, they must be treated as shown in Art. 59 provision being made for the fact that the resultant pressure for some of the bolts is not parallel to the grain. To illustrate the latter case, let it be required to

splice a $10 \times 10''$ spruce for a building to conserve the strength of the net section, taken as $8 \times 10''$, Fig. 26*m*. Allowable bending moment is,

$$M = Sbh^2/6 = 750 \times 8 \times 10 \times 10/6 = 100000 \text{ in.-lbs.}$$

Let us make plates and bolts of steel. For the former we will require,

$$100000/10 \times 16000 = 0.62 \text{ sq.in.}$$

We will use $10 \times 3/8''$ plates, furnishing 2.91 sq.in. net area.

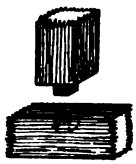


FIG. 26*n*.—Mortise and Tenon Joint.

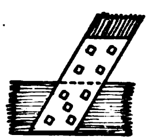


FIG. 26*o*.—Fish Joint.

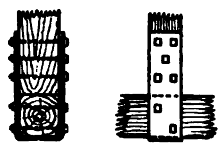


FIG. 26*p*.—Joint with Bent Connection Plate.

The stress in the plate equals 100,000/10 equals 10,000 lbs. Number of $1''$ bolts required is,

$$10000/1900 = 6,$$

arranged as shown in Fig. 26*m*. The bolts at *d* should be inserted to prevent the buckling of the plate in compression.

(4) A joint where a tension member enters a piece may be handled by:

(*a*) Mortise and tenon joint, Fig. 26*n*. This is expensive and lacks strength as an analysis will show.

(*b*) Plain fish joint, Fig. 26*o*. This joint is objectionable on account of the weakness of the bolts which are in bearing almost perpendicular to the grain and the tendency to split the horizontal piece.

(*c*) A rod as given by Fig. 25*a*. This is a good detail. Where bearing is inclined, allowable values may be interpolated according to the inclination in degrees as shown in the example below.

(*d*) Bent strap of iron or steel as represented in Fig. 26*p*. This is also an excellent detail.

Let us exemplify computations by designing rod in Fig. 25*a* to carry 10,000 lbs. when used for a roof truss of wrought iron and yellow pine. The size of the rod is

$$\pi d^2/4 = 10000/12000 = 0.83 \text{ sq.in.} \quad \text{Use } 1\tfrac{1}{16}'' \text{ dia. round rod.}$$

Taking inclination of rod with horizontal as 60 degrees.

Allowable bearing pressure $= 600 + (900 \times 30)/90 = 900$ lbs. per sq.in.

Area required $= 10000/900 = 11.1$ sq.in.

An ogee washer 4″ in diameter might be used.

(5) Where a strut enters a piece, the compression may be taken by:

(*a*) Notching, Fig. 26*q* and *r*. The former is the proper design for small stresses, acting at an inclination. It should be nailed in. Fig. 26*r* has the weakness spoken of in Art. 24 (9), and may be strengthened as there noted.

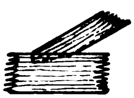


FIG. 26*q*.

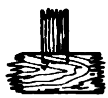


FIG. 26*r*.

Notched Joints.

(*b*) Where there is little or no horizontal component, the notching may be omitted. The strut is then held in place by toenailing or drift bolts.

(*c*) Bolting on an additional piece as shown in Fig. 26*s*.

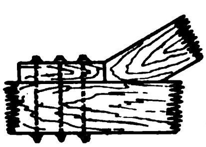


FIG. 26*s*.

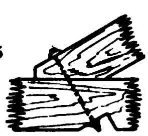


FIG. 26*t*.

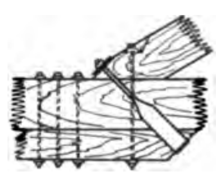


FIG. 26*u*.

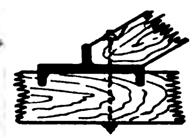


FIG. 26*v*.

Methods for Carrying Compression at an Angle.

(*d*) Bolting inclined piece as represented in Fig. 26*t*. A somewhat similar scheme, employing the strap bolt, is given in Fig. 26*u*.

(*e*) The use of cast or wrought-iron shoes. See Fig. 26*v* and 31*d*. This is probably the best method for large stresses.

Combinations of several types are frequent. However, as already pointed out, there is difficulty in making different parts work together.

As an example of the computations for these joints, let us consider Fig. 26*q* when built in red oak for a railroad bridge. Let the inclined member be a 6″×6″ at an angle of 30° with the horizontal, and let its stress be 12,000 lbs. Its horizontal component is then 10,500 and vertical 6000. At 650 and 350 lbs. per square inch respectively, there are required 16.1 and 17.1 sq.in. The depth of the notch must then be $2\frac{3}{4}$″, while the length of inclined bearing should be not less than 3″. A more exact method is indicated in Fig. 29*e*.

(6) A beam framing into another beam, usually at right angles. There are three common methods:

(*a*) Mortise and tenon, similar to Fig. 26*n*.

(*b*) Toenailing (nailing on side).

(*c*) Hangers. Two representatives of one type are shown in Fig. 25*c* and *d*.

(*a*) is expensive and weak, (*b*) is cheap and weak, while (*c*) is expensive but strong, conserving, when properly designed, the full strength of both beams.

One of the best methods is to use a built-up section. For example, an 8″×8″ timber may be made from planks, each 2×8″×16′–0″ securely bolted together with one joint every 4 feet. With good abutting joints this should be as strong as an 8″×8″ in compression. In tension it should have a net area of about 6″×6″. Also joints (4) and (5) may be designed by making one member double and passing the other through it.

Art. 27. Design of Timber Structures

The objects of design are:

To provide a structure safe under any probable circumstances.

To do this as economically as possible.

The latter is fulfilled by making the sum of the following annual charges a minimum:

(*a*) Interest on first cost.

(*b*) Maintenance.

(*c*) Sinking fund. At the expiration of the life of the structure, this fund must be sufficient to rebuild it.

(*d*) Operation.

Suppose two alternative schemes proposed for a drawbridge to be as follows: No. 1 will cost $200,000; repairs, painting, and so forth, $400 per year; bridge is estimated to last 30 years; it will be operated by 9 men at a total cost of $20 per day. For Scheme No. 2, the corresponding quantities are: cost, $150,000; repairs, $300; duration, 25 years; operation, 12 men, $27. Taking masonry as alike in both cases and interest at 4%, we may compare as follows:

Scheme.	*a.* Interest.	*b.* Maintenance.	*c.* Sinking Fund.	*d.* Operation.	Total Annual Charge.
1	$8000	$400	$3560	$7300	$19,260
2	6000	300	3600 See Trautwine, p. 46.	9850	19,750

Our analysis shows a slight preference for No. 1.

It is necessary for a student to obtain an elementary knowledge of many subjects and time can seldom be afforded for alternative designs. However, in practice, several should be drawn up on detail paper and thoroughly examined to eliminate all waste and weaknesses. Then, after comparison of costs as above outlined, the best is selected for the finished drawing of the proposed structure.

To compare wood and iron, let us take their cost when fabricated at $50 per M and 3 cents per pound.

1 sq.in. of iron will carry 12,000 lbs. tension and will cost 10 cents per foot.

27 sq.in. of wood will carry 12,000 lbs. tension and will cost 11 cents per foot.

2 sq.in. of iron will carry 12,000 lbs. compression and will cost 20 cents per foot.

15 sq.in. of timber will carry 12,000 lbs. compression and will cost 6 cents per foot.

The above is quite rough since allowance has to be made for excess of gross over net area, the reduction of allowable stress in compression, and so forth. It indicates clearly why

wood is cheaper than iron in first cost and also why the latter is often employed in tension members.

In the preceding articles, iron has been freely used in framing joints. Its shearing and bearing strength, large as compared with wood, make it particularly valuable. The objections are:

(1) It is largely blacksmith's work and therefore expensive. (Art. 42.)

(2) Small pieces are likely to be lost in shipping. (Art. 48.)

(3) In the case of error or change in design, iron is not as easy to alter as timber.

(4) Another material adds to the difficulty of handling the job. However, its advantages are such that it is used considerably.

If much framing is required and the timbers are small, spruce, white pine, Norway pine, or hemlock may be employed. All frame easily but the latter is often weak and treacherous. For heavy pieces and small amounts of notching and cutting, use yellow pine and white oak. These are strong woods but they frame with some difficulty. For long pieces, take yellow or Oregon pine. Sleepers and posts are made of cedar, chestnut, and cypress. Keys and fish plates, if of wood, are commonly of white oak.

On account of the rule that safety of the structure must not depend upon friction, bolts and screws make a much better design than spikes or nails. One of the former must be used when in direct tension and they are preferable in shear. Drift bolts, dowel pins, and nails may be employed where the stress is carried mainly in bearing. A cheap and rapid method for temporary construction is the fastening together by spikes. A disadvantage, particularly with wire spikes, is the lessened salvage value. Lag screws are difficult to drive in hard woods like oak.

In Europe, labor is cheap and lumber dear, hence elaborate joints are often made to save material. Here the reverse is true, therefore plain joints with ample connecting plates are economical.

Ties exposed to the wind and compression members must be thoroughly braced. It is sometimes necessary to use knee

bracing, Fig. 27*a*, but it is not as strong as the X bracing in Fig. 27*b*, and it introduces large bending stresses. Well-nailed boarding is considered equivalent to an X bracing in its plane. It is much more efficient if laid diagonally.


FIG. 27*a*.
Knee Bracing.

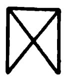


FIG. 27*b*.
X Bracing.

It is customary to make no provision for the alteration of length of timber with change of temperature and to neglect consideration of the stresses caused thereby.

Art. 28. General Description of Roof Trusses

Underneath the slate, shingles, or other covering, lies the boarding, running parallel to the peak. This boarding, or sheathing, as it is sometimes called, is planed on one side and nailed to the rafters which support it, Fig. 28*b*. The latter are "sized," that is, are notched a small amount, to bring their tops to a uniform level over the purlins. These are beams running parallel to the peak in turn resting on the trusses. They are preferably placed vertically or perpendicularly above the joint in the truss on which they rest. If otherwise located, top chord must be computed to carry the combined compression and bending.

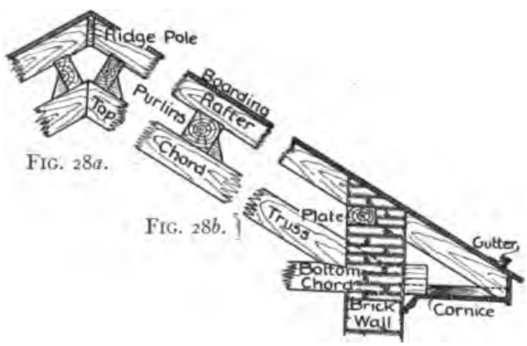


FIG. 28*c*.
Typical Arrangement of Roof Truss.

Another method is to run the boarding perpendicular to the peak and to rest it directly upon the purlins, omitting the rafters. Although more material will be used this way, it requires less work and is a better fire risk, Art. 24, (8), (*a*) and (*b*). Or both purlins and rafters may be omitted and sheathing run directly from truss to truss, resting on the top chord.

The thickness of the boarding varies from $\frac{7}{8}''$ to $3''$; it is often assumed and allowable span computed. This should be such that deflection is not more than 0.2 per cent of the same. This condition will be fulfilled by making ratio of span to thickness not more than 30. The rafters are usually $2\times4''$, $2\times6,''$ or $2\times8,''$ with larger dimension vertical. $8\times8''$, $8\times10''$, and $10\times10''$ are common for the purlins, the latter in each case being perpendicular to the roof. Two purlins are used at the peak, one on each side, Fig. 28*a*. They have the same depth as the others but only $\frac{1}{2}$ to $\frac{3}{4}$ the width on account of the lessened load. At this place, it is customary to insert between the rafters the ridgepole. It is made a little deeper to ensure full bearing and $2''$ thick. At the eaves where the truss meets the wall, the rafters are notched over a piece called the plate which lies horizontally. They are then extended to support the gutter

FIG. 28*d*.

FIG. 28*e*.

FIG. 28*f*.

FIG. 28*g*.

Forms of Roof Trusses.

and the finish around it, the three together constituting the "cornice," Fig. 28*c*. If this plate rests upon the brickwork, it is made of sufficient size to distribute the load over the masonry. A $4\times8''$ laid flat would do. In the design of the cornice, care should be taken to provide enough waterway and to exclude weather and rain from the building.

The purlins usually extend over a single "bay," as the distance center to center of trusses is called. This should be chosen to utilize some stock length of timber, such as $11'$–$6''$ to use $12'$ stuff, $13'$–$6''$, $15'$–$6''$, etc. It may be determined by conditions within the building, such as location of windows in supporting wall. When there is no such limitation, several different lengths may be tried, the cost of roof trusses estimated and the most economical chosen. This is usually about 15 feet decreasing somewhat for small spans. In a like way the most efficient arrangement for rafters, purlins, and trusses may be investigated.

Figs. 28*d*, *e*, *f*, and *g* show types of wooden roof trusses;

f representing what is perhaps the most common one. Light lines indicate those members which are usually made of iron in combination trusses, that is, in trusses made of iron and timber. However, all tension members are sometimes made of steel or iron. Like the roof systems, various trusses may be tried to secure the most favorable design. Most economical inclination is one-third pitch, that is, making the ratio of height at center to span one-third.

Bracing is usually omitted where the truss rests on brick walls. In this case the sheathing is supposed to give sufficient stiffness. Where trusses rest on isolated columns, the latter must be braced both ways. X bracing is best, but knee bracing is often used on account of the clearance required.

The dead weight per inclined square foot for various kinds of roofing exclusive of sheathing is about as follows: Shingles, 2 to 3 lbs.; slate, 5 to 8; tiles, 10 to 40; tin, 1 to 2; corrugated iron, 1 to 3; gravel, 6 to 8. The weight of timber per foot B.M. (board measure) may be estimated as, oak, 4.5. lbs.; hard pine, 4; cedar, cypress, hemlock, spruce, and chestnut, 3; white pine and poplar, 2.5. The weight in pounds, W, of the truss alone may be obtained from Jacoby's formula:

$$W=0.5\ as\ (1+0.15s).$$

Here a is the length of bay while s is the span, both in feet. The weight of snow in pounds per horizontal square foot, w, may be taken from the formula:

$$w=(l-25)\cos i,$$

where l is the latitude and i is the angle of inclination of the roof with horizontal, both in degrees. This allowance will vary somewhat with the climate. The pressure of the wind, taken as normal to the surface, may be obtained from Duchemin's formula:

$$p=C2\sin\theta/1+\sin^2\theta.$$

p=pressure in pounds per square foot, θ=angle of inclination with horizontal, and C equals a constant, a mean value for which

is 40 lbs. It will be noted that p equals C for a vertical surface.

In computations many engineers consider the resultant of vertical and perpendicular loads. This we do not favor, as the usual arrangement of the rafters takes care of component parallel to roof. A very simple method is to add directly the entire weight of roof, snow, and wind, and consider them as acting perpendicularly on every part of the roof surface. To be sure this is higher than the actual load on boarding and rafters, but it serves as an impact allowance and also to take care in a way of a concentrated load. If without knee braces, the same method may be pursued for ordinary trusses. The writer, however, considers the latter poor engineering. For knee-braced trusses, it is never allowable. The proper way then is for all framed structures to obtain maximum stress of each kind after a full consideration of all possible loadings.

Art. 29. Computations for a Roof Truss

Let us suppose trusses of the type shown in Fig. 28*f* to be of 40′ span, 13′–6″ c. to c., and angle of rafters with horizontal, 30 degrees. Roofing, slate weighing 8 lbs. per square foot. Material, spruce with wrought iron rods upset at ends for ten-

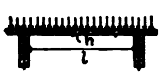


FIG. 29*a*.—Boarding.

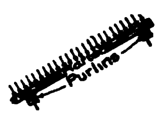


FIG. 29*b*.—Rafter.

Load Diagrams.

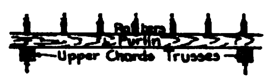


FIG. 29*c*.—Purlin.

sion members. For the computation of sheathing, rafters, and purlins, we will allow 32 lbs. for wind, 15 lbs. for snow, and 13 lbs. for dead load, making 60 lbs. per square foot, perpendicular to the roof. Note that the actual computation below shows the dead load to be 15.5 lbs. per square foot. See Fig. 29*f* for finished design.

(1) Allowable distance center to center of rafters, Fig. 29*a*.

$$S_t = 6M/bh^2 = 3wl^2/4bh^2 \quad \text{or } l = 2h(bS_t/3w)^{\frac{1}{2}}.$$

Let us try $\frac{7}{8}''$ boards. Then, taking a strip $12''$ wide, $h=\frac{7}{8}''$, $b=12''$, $w=5$ lbs. per linear inch, $S_f=750$ lbs. per square inch, hence $l=42.9''$. To prevent excessive deflection, this distance must be reduced. $\frac{7}{8}''\times 30=26.2$, we will make it 6 rafters to a bay equals $27''$ spacing.

(2) Size of rafter (Fig. 29*b*). Make $2''$ wide.

$$S_f=3wl^2/4bh^2 \quad \text{or} \quad h=0.5l(3w/bS_f)^{\frac{1}{2}}.$$

$$\text{Span}=l=240 \text{ sec. } 30 \text{ deg.}/3=92.4''.$$

$$w=60\times 27/(12\times 12)=11.25 \text{ lbs. per linear inch.}$$

$$S_f=750 \text{ lbs. per square inch.}$$

Substituting, $h=6.93''$, and we will use $2''\times 8''$, the next size above.

(3) Size of purlin (Fig. 29*c*). Carpenter usually arranges

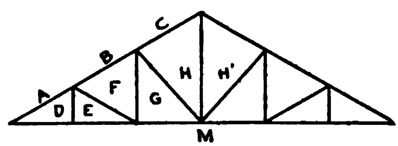


FIG. 29*d*.

rafters to suit himself. Most unfavorable case is shown in the figure. Its maximum effect is the same as that of a uniformly distributed load of equal amount.

$$S_f=3wl^2/4bh^2 \quad \text{or} \quad bh^2=3wl^2/4S_f.$$

$$l=162'', S_f=750 \text{ lbs. per square inch.}$$

$$w=92.4\times 60/(12\times 12)=38.5 \text{ lbs. per linear inch.}$$

Hence,

$$bh^2=1010. \quad \text{Use } 10''\times 10'', bh^2=1000.$$

(4) Size of members. Estimated weight of roofing is: slate 8 lbs., sheathing 2.5 lbs., rafters 2 lbs., purlins 3 lbs.; total

15.5 lbs. per inclined square foot. Estimated dead weight of truss is $\frac{1}{2}$ $13.5\times40(1+0.15\times40)=1890\#$. Dead panel load $=\frac{1890}{6}+15.5\times13.5\times7.7=1920\#$. Panel load for snow $=15\times13.5\times40/6=1350\#$. Wind per inclined square foot is $40\times2\sin 30\text{ deg.}/1+\sin^2 30\text{ deg.}=32\#$. Wind apex load $=32\times7.7\times13.5=3340\#$. Consider truss as fixed at both ends.

Member.	Total Stresses in Kips.*					In Inches.					Remarks.
	Dead	Snow	Wind *L.*	Wind *R.*	Max.	Allow. Unit Stress.	Area Requir'd	Use.	Area Furnish'd	Excess.	
AD	C 9.60	C 6.75	C 8.70	C 5.74	C 25.05	530	47.2	8″×8″	64.0	16.8	$l=92''$
BF	C 7.68	C 5.40	C 6.79	C 5.74	C 19.87	530	37.4	8″×8″	64.0	26.6	Same as *AD*
CH	C 5.76	C 4.05	C 4.85	C 5.74	C 15.55	530	29.4	8″×8″	64.0	34.6	"
DM	T 8.30	T 5.84	T 10.00	T 3.34	T 24.14	600	40.2	8″×10″	80.0	39.8	50% for joints
EM	T 8.30	T 5.84	T 10.00	T 3.34	T 24.14	600	40.2	8″×10″	80.0	39.8	"
GM	T 6.64	T 4.66	T 6.67	T 3.34	T 17.97	600	30.0	8″×10″	80.0	50.0	Same as *DM*
EF	C 1.92	C 1.35	C 3.82	0	C 7.09	462	15.3	4″×6″	24.0	8.7	$l=92''$
GH	C 2.56	C 1.80	C 5.07	0	C 9.43	478	19.7	6″×6″	36.0	16.3	$l=122''$
DE	0	0	0	0	0	12000	0.0	1 rd. $\frac{5}{8}''$	0.31	0.31	Minimum size
FG	T 0.96	T 0.68	T 1.91	0	T 3.55	12000	0.30	1 rd. $\frac{5}{8}''$	0.31	0.01	Upset at ends
HH'	T 3.84	T 2.70	T 3.83	T 3.83	T 10.37	12000	0.87	2 rds. $\frac{3}{4}''$	0.88	0.01	"

(5) Joint *ADM*. Make like Fig. 26*v*. Bearing area required for the stress in *DM* is $25.05\cos 30^\circ/1000=21.7$ sq.in. Use two notches, each 8″ wide by $1\frac{3}{8}''$ deep, affording 22.0 sq.in. bearing area. Shearing length required is, $25.05\cos 30^\circ/8\times80=34.0''$. Make two of 1′–6″ as shown in Fig. 29*f*. For stress in *AD*, $25.05\cos 30^\circ/770=28.2$ sq.in. Use $3\frac{1}{2}''\times8''$ bearing. Allowing 3000 lbs. per sq.in. shear, necessary thickness of casting is, $25.05\cos 30^\circ/3000\times8=0.91''$. Using an allowable flexural stress of 5000 lbs. per sq.in.,

$$t=(3Wl/bS_f)^{\frac{1}{2}}.$$

Here, $W=25.05\cos 30^\circ=21{,}700$ lbs., $l=3.5''$, $b=8''$, $S_f=5000$ lbs. per sq.in. Substituting, $t=2.39''$. In Fig. 29*f*, casting is

* An abbreviation for kilo pounds, that is, thousands of pounds.

made 1⅛″ thick. Since required thickness varies as distance from the top, the lower half would be deficient in strength. The filleting of the corners of the casting, Art. 16, would help some by increasing strength of iron and lowering point of application of resultant pressure. Also stiffening ribs might be employed. In any event, supplement design as shown with a

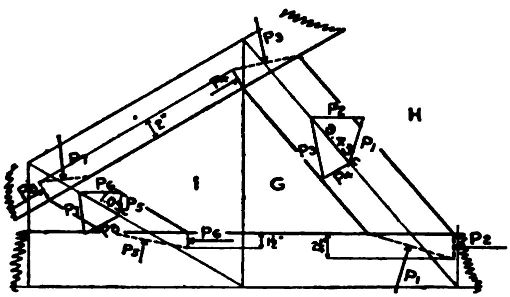


FIG. 29e.

special detail of the casting, giving necessary dimensions at important points.

(6) Area washers.

For *DE* and *FG*, 3550/300 = 11.8 sq.in., use 4″ O. G. Washer.
For *HH′*, 10370/300 = 34.6 " " 4.5″×8″ Washer, special.

(7) Notches for *EF* and *GH*. (Fig. 29e.)

	Total Pressure in Kips.	Area Sq. in.	Pressure in Lbs. per Sq. In.	Inclin. with Grain. Degrees.	Allowable Pressure. Lbs. per Sq. In.
P_1	7.40	49.8	148	74	430
P_2	8.20	13.5	610	49	620
P_3	9.80	38.4	255	72	440
P_4	4.80	12.0	400	79	390
P_5	3.60	48.7	75	79	390
P_6	6.90	9.0	770	30	770
P_7	6.80	30.0	230	66	490
P_8	6.20	12.0	510	60	530

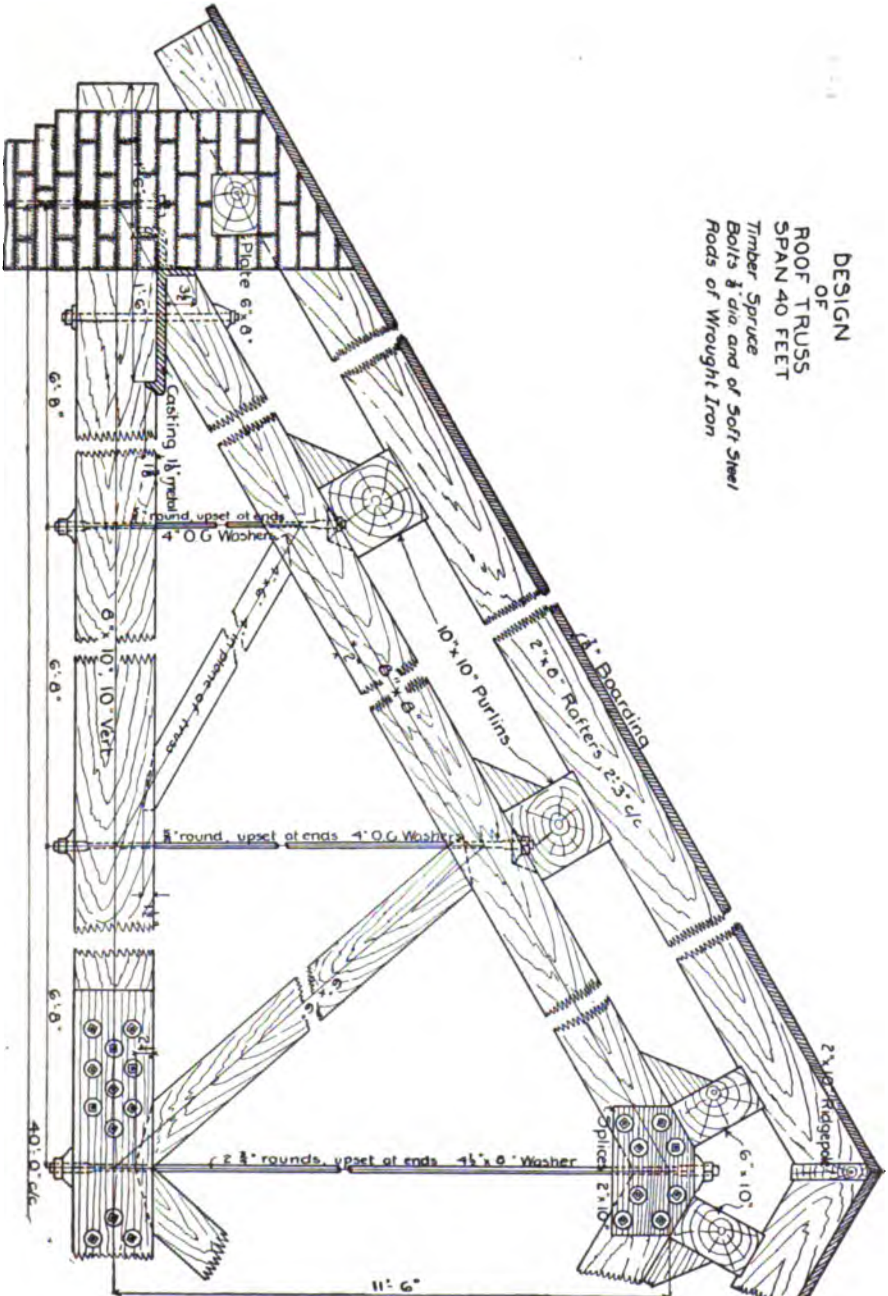

DESIGN
OF
ROOF TRUSS
SPAN 40 FEET
Timber Spruce
Rods of Wrought Iron
Plate 6"×8"
Casting
4" O.G. Washers
10"×10" Purlins
Boarding
Splice
Ridgepole
6"×10"
6'-8"
6'-8"
6'-8"
11'-6"
40'-0" c/c

FIG. 29f.

(8) Splice at center. Use $\frac{7}{8}''$ steel bolts and two fish plates, each $3'' \times 10''$. Value of bolts, Art. 26, is 2000 lbs. each. Number required is $17,970/2000 = 9$.

Art. 30. Trussed Beams *

Figs. 30*a* and *c* show the styles known as the king and queen post truss respectively. They are trussed beams only when horizontal chord is continuous. Computations often assume them to be jointed structures. Where the angle of inclination, θ, is $20°$ or more, with horizontal, the error will not be a serious one. We sometimes find them inverted, but the depth is then made such that they need not be considered as trussed beams. Taking the usual case of constant sections and uniform load, the correct method for either of the above trusses is as follows:

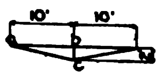


FIG. 30*a*.—King Post Truss.

(1) Assume a jointed structure and determine sections. This assumption is for the purpose of obtaining sizes and is carried no farther.

(2) Compute the deflection at panel points caused by any uniform load, W_1, acting on entire length of top chord, considered as a simple truss. Reactions must be obtained by treating beam as continuous.

(3) Compute the uniform load, W_2, which will cause equal deflections at panel points when the top chord is considered a simple beam for its entire span.

(4) Then, for any uniform load, $W_2/(W_1+W_2)$ of that load acts as if on a simple beam of length equal to the span. $W_1/(W_1+W_2)$ of that load acts as in a continuous beam.† From the reactions for the same, stresses for the truss may be determined.

(5) Those in the beam are then the result of compression due to truss action, plus bending due to $W_2 \times \text{Load}/(W_1+W_2)$

* See "Modern Framed Structures," by Johnson, Bryan, and Turneaure, Part II, p. 408. While theory given here bears a strong resemblance to that in above reference, it was consulted only after manuscript was finished.

† This follows from the fact that deflections due to truss action and that due to beam action must be the same at panel points.

on simple beam of span length, plus bending due to $W_1 \times$Load/(W_1+W_2) on a continuous beam with supports at each panel point of the truss.

(6) If stresses so found are close to allowable values, details may be determined for the final design; otherwise, revise and recompute.

As an example, let us take a beam whose span is 20 feet with a depth at center of 2 feet as shown in outline in Fig. 30*a* and as detailed in Fig. 30*b*. Let the load be 2000 lbs. per lineal foot.

(1) Assuming discontinuity in top chord, stresses are; *bc*, 20,000 # *C*; *ab*, 50,000 # *C*; *ac*, 51,000 # *T*; bending in *ab*, $20{,}000 \times 120/8 = 300{,}000$ in.lbs. Use yellow pine. Allowable compression, $1200-15l/d$; flexural stress, 1500; bearing 600 and 1500; E, 1,600,000. For iron, 10,000 in tension and $E=25{,}000{,}000$, all in pounds and inches.

For *bc* we will try a 4″×12″, larger than necessary but affording a good design. If we take the top chord as made of two timbers, each 8″ wide by 10″ deep, its stresses will be

$$50{,}000/160+6\times 300{,}000/16\times 10\times 10 \quad = 1437 \text{ lbs. per sq.in.}$$

Required area of rod $= 51{,}000/10{,}000 = 5.1$ sq.in.

Use one round, 2 9/16″ dia., area $= 5.16$ sq.in.

(2) Deflection due to a uniform load of 1000 lbs. Load on truss, 625 lbs. Everything in pounds and inches.

Member.	S Stress.	l Length.	A Area.	E Mod. Elas.	Sl/AE
ab	1562	240	160	1,600,000	2.29
bc	625	24	48	1,600,000	0.12
ac	1593	245	5.16	25,000,000	4.83
				Total......	7.24

Deflection equals $7.24/625 = 0.0116''$.

(3) Deflection $= 0.0116 = \dfrac{5W_2 \times 240^3 \times 12}{384 \times 1{,}600{,}000 \times 16 \times 10^3}$.

Hence $W_2 = 137$ pounds.

(4) Of this uniform load of 2000 pounds per lin. ft., 137/1137 equals 240 lbs., acts as on a simple beam, while 1760 lbs. affects truss. Stresses in the latter are:

$$ab = 55{,}000\,\#\,C, \qquad bc = 22{,}000\,\#\,C, \qquad ac = 56{,}100\,\#\,T.$$

Unit stress in bc equals 22,000/48 = 460 lbs. per sq.in. C.

" " ac " 56,100/5.16 = 10,900 " " " T.

(5) Max. moment in ab occurs where the shear equals zero. This happens at center and also at $R/w = 9000/2000 = 4.5$ feet from either support.

For the former, $M = 9000 \times 10 - 20{,}000 \times 5 = -10{,}000$ ft.lbs.

For the latter, $M = 9000 \times 4.5 - 9{,}000 \times 2.25 = 20{,}250$ ft.lbs.

Stress in $ab = -55{,}000/160 \pm 6 \times 20{,}250 \times 12/(16 \times 10 \times 10)$

$-344 \pm 911 = 1255C$ or $567T$ in lbs. per sq.in.

(6) The stress in ac is greater than that allowable. As we make this member larger, still more of the load will come on the truss. About a 2 11/16″ rod is required, we will make it $2\frac{3}{4}''$ diameter and recompute. For the deflection in (2), we get .0106″. The uniform loads on beam and truss are 220 and 1780 lbs. respectively, and total stresses are,—

$$ab = 55{,}750\ C, \qquad bc = 22{,}300\ C, \qquad ac = 56{,}900\ T.$$

The unit stresses are, in lbs. per sq.in.,—

$$ab = 1236\ C, \qquad bc = 465\ C, \qquad ac = 9580\ T.$$

A 2 11/16″ rod might have done but ab and bc have reserve

strength and it is best to have a little in *ac*. It would not be advisable to stress *ab* the full allowable flexural value of 1500 lbs., as it is in part a compression member. Use casting at *c* designed for a load of 22,300 lbs, concentrated below, and uniformly distributed over 12″ above. Bearing values must be tested at shoulders on *bc* and for rods at *a*.

Problems of this kind may often be advantageously solved by the aid of the method of least work, This principle, with which the student should already be familiar, states in brief that the load is so divided among statically indeterminate

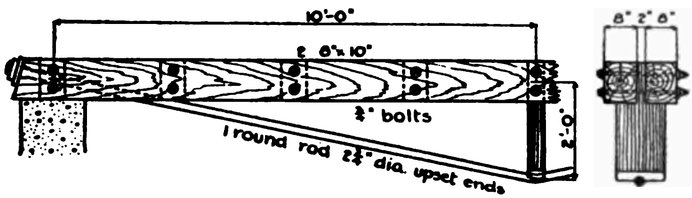


FIG. 30*b*.—Design of King Post Truss Shown in Fig. 30*a*.

systems as to make the sum total of the work a minimum. Let us then apply it to this case.

Let E_1, E_2, E_3, be moduli of elasticity for *ab*, *bc*, and *ac* resp.
A_1, A_2, A_3, be areas for *ab*, *bc*, and *ac* resp.
L_1, L_2, L_3, represent spans *ab*, *bc*, and *ac* resp.
I_1 be the moment of inertia of *ab* about an horizontal axis through the center of gravity.
P be the load on the truss.
$W = 2wL_1 =$ total load.
$\theta = \tan^{-1} bc/ab$.

Further, let M equal the bending moment in beam at a point distant x from either reaction, and let K represent the work done. Then

$$K = 2\int_0^{L_1} M^2 \delta x/(2EI) + \tfrac{1}{2}\Sigma S^2 L/AE,$$

where S designates the total stress in bars. Now substituting for S, the stresses in the different bars, and for M, $\frac{W-P}{2}x - \frac{1}{2}wx^2$,

$$K = \frac{1}{E_1I_1}\left(\frac{W^2L_1^3}{12} - \frac{PWL_1^3}{6} + \frac{P^2L_1^3}{12} - \frac{WwL_1^4}{8} + \frac{wPL_1^4}{8} + \frac{w^2L_1^5}{20}\right)$$
$$+\frac{1}{2}\frac{P^2L_1 \tan\theta}{A_2E_2} + \frac{P^2L_1}{4}\left(\frac{\csc^2\theta \sec\theta}{A_3E_3} + \frac{\cot^2\theta}{A_1E_1}\right).$$

Following the rules of calculus, we differentiate this expression with regard to P and place it equal to zero to get the minimum value of K, and obtain

$$P = \frac{5WL_1^2}{48E_1I_1\left(\frac{L_1^2}{6E_1I_1} + \frac{\tan\theta}{A_2E_2} + \frac{\csc^2\theta \sec\theta}{2A_3E_3} + \frac{\cot^2\theta}{2A_1E_1}\right)}.$$

Substituting values given in problem above, we compute P to be 22,200 lbs., in substantial agreement with the method of deflections.

By putting an initial camber, that is, an upward curvature in the stringer, its stresses may be changed a considerable amount. The deflection in the problem just considered is $0.0106 \times 40{,}000/1125^* = .38''$, about $\frac{3}{8}''$. If now the nuts at the end be screwed up before the load is put on so that it has a camber of $0.38 \times 2000/1780 = 7/16''$, the load acting as a simple beam will be zero.

Any load for the king post truss or a symmetrical load for the queen post truss may be similarly treated. For the latter under unsymmetrical loads, above analysis will not hold, since horizontal component of stress in bottom chords must be constant. We proceed as follows:

(1) Assume jointed structure with a diagonal, ce, compute stresses and determine sections making

$$ab = bc = cd, \quad ae = ef = fd, \quad \text{and} \quad be = cf.$$

* Recomputed value for 1137 in (4).

This assumption is for the purpose of obtaining sizes and is carried no farther.

(2) Taking *ad* as a simple beam, compute deflections at *b* and *c* caused by given unsymmetrical loads, and call the mean of these, D_1.

(3) Still considering *ad* as a simple beam, compute upward deflection, d_2, at either *b* or *c* due to two equal loads, *P*, at *b* and *c*.

(4) Taking Fig. 30c as a truss, find downward deflection, d_3, due to the loads *P* at *b* and *c*.

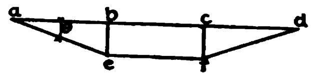


FIG. 30c.—Queen Post Truss.

(5) Note that the truss offers no resistance to an upward movement at *b*, accompanied by an equal downward movement at *c*. Also that the average deflection at panel points, D_1, due to whole load acting on beam of entire span, minus upward deflection, D_2, of beam due to reactions, *T*, carried by truss, equals downward deflection, D_3, at same points due to the truss action. That is, $D_1 - D_2 = D_3$. But, because of the proportionality of deflections, $D_2 = Td_2/P$ and $D_3 = Td_3/P$. Substituting, $T = PD_1/(d_2 + d_3)$.

(6) The stresses in the truss may then be determined. In the top chord, besides its direct compression, it will be a simple beam of length *ad* subject to the given loads plus the two upward ones, *T*, at *b* and *c* just obtained.

(7) If stresses are satisfactory, problem is completed; if not, revise and recompute.

Art. 31. Description of Bridges

FIG. 31*a*.
Queen Post Truss.

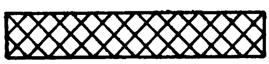


FIG. 31*b*.
Lattice Truss.

FIG. 31*c*.
Howe Truss.

The above represents the leading types of bridge trusses: *a*, the queen post truss mentioned in the preceding article; *b*, the lattice; and *c*, the Howe: the latter is the best and most common form.

There are usually two similar trusses. Between them are the floorbeams, generally having the larger dimension for their depth. On top of these floorbeams and parallel with the truss, are stringers with a depth two to six times their width. On these are laid the ties in a railroad bridge or the plank in a highway bridge.

Greater economy may be obtained by the use of deep stringers but 16″ is about the greatest depth that can be easily obtained and its width must not be less than one-sixth the depth on account of the tendency of narrow beams to buckle.

In a highway bridge, the dead floor load should be computed as it varies a great deal with the width. For railroad single track bridges, 400 lbs. per lin.ft. may be used for the weight of one track, and one-fifth the live load for the weight of floor. The weight of the trusses for any wooden bridge may be taken as their live load burden times one three-hundredth of the span in feet. Here the weight is for the same number of trusses and for the same length as the load. If we take live weight in pounds on each lineal foot of truss, we obtain weight of truss in pounds per lineal foot.

In a railroad bridge, ties are usually spaced 12″ on centers, are 9 to 12 feet long, and 6×8″ in section with the latter vertical. However, they should be tested for their loads. About nine inches inside of each rail are placed the guard rails, both these and the main rails being spiked to each tie. Approximately 20″ center to center outside of main rails are placed the guard timbers, about 6″ vertical by 8″, which are bolted to every third tie.

Six stringers commonly support the latter. Of these, two are placed under each rail and are computed to carry the loads therefrom. The other two are put at either end of the tie. The stringers rest on the floorbeam, these in turn rest on the bottom chord or are hung from the verticals. When the floorbeams are not placed at a joint, the chord must be designed to carry the bending moment as well as the direct stress.

In a Howe truss, verticals are ordinarily of iron or steel and are upset at their ends. Special washers of wood or iron are often necessary to take the stress into the chords, these washers being figured to carry their loads when uniformly dis-

tributed. The verticals usually pass through the block of oak or cast iron on which the diagonals rest, and also through the bottom chord.

If cast iron be used for the angle block, it need not be finished. The lugs at the top, Fig. 31*d*, should have a hole for a bolt to fasten in the diagonals. Those at the bottom should be figured to furnish sufficient bearing area on timber. The thickness of the cast iron at this point should suffice for the shear and moment. Somewhat similar castings are placed at the bottom of the top chord and they may also be used as bearing blocks for the laterals.

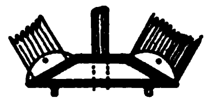


FIG. 31*d*.—Angle Block Howe Truss.

These are best kept in the plane of the chords. X bracing should be employed at the panel points of deck bridges, with knee bracing at the entrance to through bridges.

Chords are often made of three or more pieces. They are spliced as near a joint as possible. If compound, pieces should be kept at least two inches apart and occasional wooden fish plates inserted between them. Splices should be arranged to stagger, that is, they should be so arranged that no two will occur at same or nearby points. Diagonals are usually of two timbers, while between them run the counters of one piece each, the two sets being tightly bolted together at their intersection.

No provision is made for the expansion due to the change of temperature.

Art. 32. Computations for a Bridge

(See Figs. 32*d* and *e*)

Let it be required to design a through Howe truss for a single track railroad. It is divided into eight panels of 12′–0″ each making a total of 96′–0″. We will make distance center to center of trusses 16′–0″, thus providing the necessary 13′–0″ in the clear. Headroom above base of rail should be 20′–0″, hence center to center of chords will be made 24′–0″. The live loading is 4000 lbs. per lin.ft. plus a concentration

of 8000 lbs. for floor system only, both per track. For wind load, use 100 and 300 lbs. per lin.ft. on top and bottom chords respectively, the latter to be treated as live. Material, hard pine with wrought iron tension members.

(1) Size of stringers. Using four floorbeams to a panel length, the span of the stringer becomes three feet. Allowing 200 and 100 lbs. per lin.ft. per rail for track and stringer respectively, the latter is a beam subjected to a uniform load of 6900 lbs. and a concentrated load of 4000 lbs. The max-

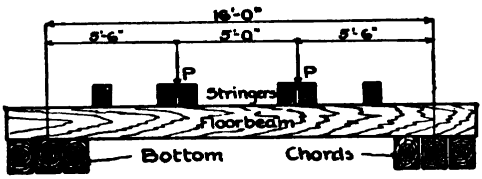


FIG. 32*a*.

imum shear is then 7450 lbs. and maximum moment 67,000 in.lbs.

Least allowable value for shear, $bh = 1.5 \times 7450/70 = 160$
" " moment, $bh^2 = 6 \times 67{,}000/1000 = 402$

These requirements will be satisfied by using under each rail two $8'' \times 10''$ with the latter vertical.

(2) Size of floorbeam, span 16', Fig. 32*a*.

$$P = 2300 \times 3 + 4000 = 10{,}900 \text{ lbs.}$$

Uniform load, estimated, $100 \times 16 = 1600$ lbs.
Maximum shear, 11,700 lbs. Maximum moment, 758,000 in.lbs.

Required in shear, $bh = 1.5 \times 11{,}700/70 = 250.$
" " moment, $bh^2 = 6 \times 758{,}000/1000 = 4548.$

Use two $9'' \times 16''$, former horizontal.

(3) Loads.

Dead load per foot per truss for track = 200 lbs. on bottom chord.
" " " floor = 400 " "
" " " truss = $2000 \times 96/300$.
= 640 lbs., $\frac{1}{2}$ top, $\frac{1}{2}$ bottom.

Dead upper panel load is $0.32 \times 12 =$ 3.8 kips.
" lower " " $0.92 \times 12 = 11.0$ "
Live lower " " $2.00 \times 12 = 24.0$ "
Wind upper " " $0.10 \times 12 =$ 1.2 "
" lower " " $0.30 \times 12 =$ 3.6 "

(4) Stresses. Values are in kips and kip in.
(See figure below.)

-	Member.	Dead.	Live.	Wind.	Maximum.
	U_1U_2	25.9 C	42.0 C	....	67.9 C
Upper chord...	U_2U_3	44.4 C	72.0 C	2.2 C	118.6 C
	U_3U_4	55.5 C	90.0 C	3.5 C	149.0 C
	L_0L_1	25.9 T	42.0 T	12.6 T*	80.5 T
Lower chord...	L_1L_2	44.4 T	72.0 T	19.3 T*	135.7 T
	L_2L_3	55.5 T	90.0 T	23.3 T*	168.8 T
	L_3L_4	59.2 T	96.0 T	24.6 T*	179.8 T
	U_1L_0	57.9 C	94.2 C	9.1 C	161.2 C
Diagonals.....	U_2L_1	41.4 C	70.5 C	....	111.9 C
	U_3L_2	24.8 C	50.3 C	....	75.1 C
	U_4L_3	8.3 C	33.6 C	....	41.9 C
	U_1L_2	41.4 T	3.4 C	....	....
Counters......	U_2L_3	24.8 T	10.1 C	....	....
	U_3L_4	8.3 T	20.2 C	....	11.9 C
	U_1L_1	48.0 T	84.0 T	....	132.0 T
Verticals......	U_2L_2	33.2 T	63.0 T	....	96.2 T
	U_3L_3	18.4 T	45.0 T	....	63.4 T
	U_4L_4	11.0 T	30.0 T	....	41.0 T

* Three kips is due to the effect of the wind on the upper chord.

Stresses in portal. See Fig. 32*c*.

In *a*, $2.1 \times 13.4 \times 1.414/5 = 8.0$ T or C.
Max. moment in U_1L_0 is $3.5 \times 5.0 \times 12 = 210$.
" " U_1U_1 or L_0L_0 is $2.1 \times 60 = 126$.
" " bottom chord. See Fig. 32*b*.*
$18.5 \times 54 - 8.0 \times 36 = 711$.

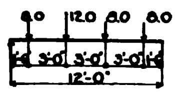


FIG. 32*b*.—Bottom Chord.

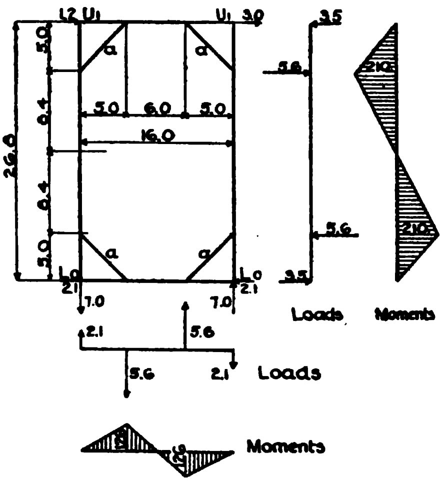


FIG. 32*c*—Portal.†

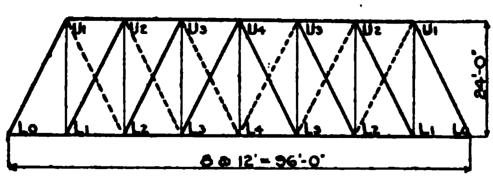


FIG. 32*d*.—Diagram Howe Truss.

* Two kips has been added here to cover weight of bottom chord and extras and to allow of possible misplacement of floor beams from assumed position.

† See Chap. VII, Vol. II, for explanation.

(5) Table of unit stresses.

Member.	Max. Kips.	Try Hor. Vert.	Unit Stresses in Lbs. per Sq. in. Direct.	Secondary.	Total.	Unsupported Length. Inches.	Allowable.
U_1U_2	67.9 C	12″×10″	566	...	566	144	656
U_2U_3	118.6 C	20″×10″	593	...	593	144	656
U_3U_4	149.0 C	24″×10″	621	...	621	144	656
L_0L_1	80.5 T	30″×16″	168	556	724	...	800
L_1L_2	135.7 T	34″×16″	250	490	740	...	800
L_2L_3	168.8 T	38″×16″	277	440	717	...	800
L_3L_4	179.8 T	38″×16″	297	440	737	...	800
U_1L_0	161.2 C	26″×12″	516	155	671	161	666
U_2L_1	111.9 C	Two 10″×10″	560	...	560	161	639
U_3L_2	75.1 C	" 8″× 8″	586	...	586	161	600
U_4L_3	41.9 C	" 6″× 8″	436	...	436	161	530
U_1L_2	....	One 6″× 8″	...	...	...	161	530
U_2L_3	....	" 6″× 8″	...	...	...	161	530
U_3L_4	11.9 C	" 6″× 8″	248	...	248	161	530
U_1L_1	132.0 T	Two 3½″ rounds	6860	...	6860	...	7000
U_2L_2	96.2 T	" 3″ "	6800	...	6800	...	7000
U_3L_3	63.4 T	" 2½″ "	6450	...	6450	...	7000
U_4L_4	41.0 T	" 2″ "	6530	...	6530	...	7000
Top diags..	3.8 C	One 6″× 6″	110	...	110	120	600
Top ties....	1.8 T	" ¾″ round	4100	...	4100	...	7000
U_1U_1	6.7 T	One 8″×12″	70	660	730	...	800
a, Fig. 32*c*	8.0 T	" 4″× 6″	333	...	333	...	800
	8.0 C		333	...	333	84	590
Bott. diags.	15.8 C	" 6″× 6″	440	...	440	120	600
L_1L_1	9.4 T	One 1⅜″ round	6350	...	6350	...	7000
L_2L_2	6.7 T	" 1⅛″ "	6770	...	6770	...	7000
L_3L_3	4.5 T	" 1″ "	5720	...	5720	...	7000
L_4L_4	2.7 T	" ¾″ "	6140	...	6140	...	7000
L_0L_0	11.0 T	" 8″×12″	110	660	770	...	800

In the design, Fig. 32*e*, the bridge must be built to conform to the above results. The stress in wooden tension members is kept low to allow for the joints. In computing U_1L_0 for

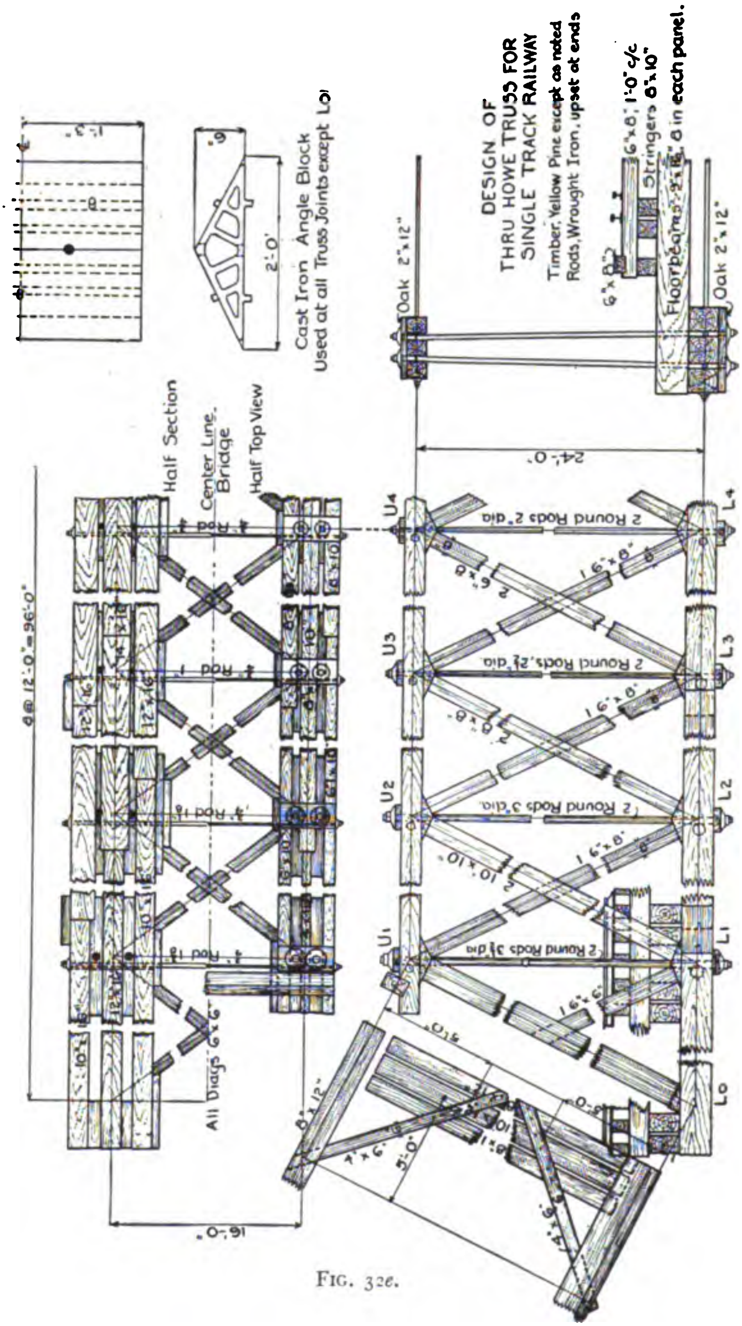

DESIGN OF
THRU HOWE TRUSS FOR
SINGLE TRACK RAILWAY
Timber, Yellow Pine except as noted
Rods, Wrought Iron, upset at ends
Cast Iron Angle Block
Used at all Truss Joints except L0
Half Section
Center Line Bridge
Half Top View
8 @ 12'-0" = 96'-0"
24'-0"
16'-0"
All Diags 6"x6"
2 Round Rods 2" dia.
2 Round Rods 2½" dia.
2 Round Rods 3" dia.
Oak 2"x12"
Floorbeams

FIG. 32e.

moment, it is considered as a solid beam, a little different from its actual construction but probably agreeing closely with its strength. It is better to nail diagonal batten plates of say one inch stuff along top and bottom, leaving small spaces between for ventilation. For L_0L_0 the floorbeams are used as they will be amply strong. Bolts and other details are not shown in the drawing. At all joints, packing blocks should be used and pieces thoroughly bolted together. In built up compression members, care should be used that the slenderness ratio for individual members does not exceed that for the column as a whole.

Art. 33. Trestle Bents.* (Fig. 32f)

While these may be computed and designed like any other structure, they are usually built from standard plans evolved from experience. Standardization is rendered necessary by the need for keeping in stock the timber for renewal.

The ties, rails, and guard timbers should be the same as for a Howe truss. Stringers are usually built of three timbers,

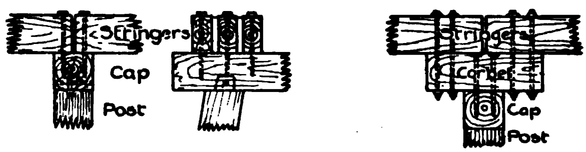


FIG. 33a. FIG. 33b.
Details at Cap.

each 6 to 8″ wide and 14 to 18″ deep. They are well bolted together and packed by fish plates at caps and spools (cast-iron washers), elsewhere to keep them 2″ apart. The span varies from 12 to 16 feet. Separate timbers may extend over one or two spans, in the latter case breaking joints. They may rest directly on top of the cap of the bents as shown in Fig. 33a, in which case they are drift bolted to it. Or "corbels" may be placed underneath as seen in Fig. 33b, when it is

* See "Theory and Practice of Modern Framed Structures," by Johnson, Bryan, and Turneaure, 1904 ed., Chap. XXV.

bolted to stringers and drift bolted to the cap. Corbels add to the cost, contribute to shrinkage and decay, and should be avoided.

The structure which supports the stringers is called a "bent." These are usually built about as represented in Fig. 33*c*. The posts are under or nearly under the rail, while the battered posts are placed just outside at the top. The inclination of the latter is 2 to 3″ per foot. Sometimes a square bent is used, but it is suitable only for tangents and small heights, Fig. 33*d*.

The cap may be made whole as shown in Fig. 33*a* and *b*. In this case, it is usually about 12″×12″×10′ to 14′. Or the posts can be mortised into the cap as seen in Fig. 33*e*. In

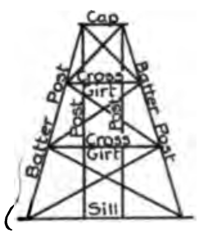


FIG. 33*c*.
Typical Bent.

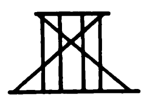


FIG. 33*d*.
Square Bent.

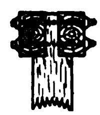


FIG. 33*e*.
Post Mortised into Cap.

the latter event, two 6″×12″ are firmly bolted to the posts, which are notched to receive them. These posts are usually 12″×12″ mortised or doweled into the sill, also a 12″×12″ extending 2 or 3 feet outside of the joint. The sills rest on pile or masonry foundations, preferably being firmly bolted thereto.

The bracing in the plane of the bent is called "sway bracing" and may be omitted if batter posts are used and if the height of the bent does nor exceed 20 feet. Each diagonal should be 2 or 3″×12″, firmly bolted at each end, and spiked at each of the other posts. Cross girts, horizontal transverse members, may be in pairs bolted to the posts; or single members, framed in between and toenailed to them. Sometimes the bents are made in stories, the cross girt forming the sill for one story and a cap for the one below it.

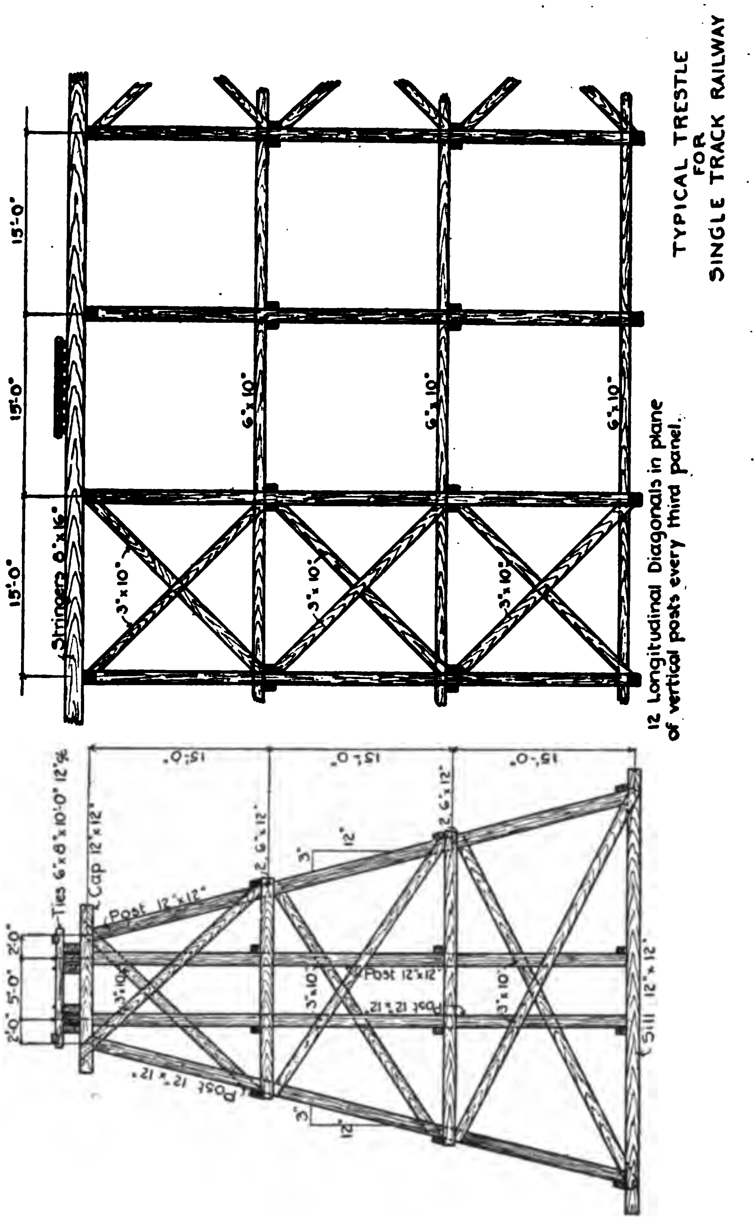

TYPICAL TRESTLE
FOR
SINGLE TRACK RAILWAY
12 Longitudinal Diagonals in plane of vertical posts every third panel.
15'-0"
15'-0"
15'-0"
Stringers 8"x16"
3"x10"
6"x10"
Cap 12"x12"
Post 12"x12"
Sill 12"x12"
2'-0" 5'-0" 2'-0"

FIG. 33*f*.

The bracing in a plane parallel to the track is called the longitudinal bracing. Diagonals are about 6″×8″ and girts about 6″×12″. It is placed in the plane of the center of track. Plank bracing in plane of vertical or battered posts is frequently used and should be just as good if an efficient connection between cap and post is obtained.

Usual size of drift or other bolts is $\frac{3}{4}''$ diameter.

Except when using standard plans, the safety of which in similar locations has been proven in practice, loads should be estimated and every piece and joint carefully proportioned not to exceed allowable limits. In designing the longitudinal bracing, stresses due to the braking of the train must not be forgotten. This may be estimated at one-fifth the total live weight. The sway bracing has to carry the wind load, a high value of which should be chosen to provide strength to resist the buckling of the posts.

CHAPTER IV

FABRICATION OF STRUCTURAL STEEL *

Art. 34. Organization of Administration

WE pass now to the design of structures largely of steel. Let us first take up the company, its men, machines, and methods by which the rolled shapes are fabricated into bridges and buildings.

The company may be divided into the following departments:

Executive,	Ordering,	Shipping,
Sales,	Operating,	Erecting,
Engineering,	Inspecting,	Financial.

In the executive department are the president, vice-president, general manager, purchaser, and treasurer. They decide upon the policy of the company, appoint minor officers, and exercise a general supervision of all employees. Those who occupy these positions should have capacity for handling men and good business judgment in addition to a technical education and a thorough knowledge of the work. However, they are not directly productive and we shall not consider them further.

The sales department endeavors to secure business for the company, that is, to obtain contracts for making bridges and buildings. In competition they submit figures based on estimated weights and drawings obtained in the engineering department. See Arts. 51, 53, and 57. If the salesmen "land" the job, the engineering department also makes the preliminary order, Art. 63, the detail drawings, Arts. 58 and 62, and the final lists of materials, Arts. 63, 64, and 65.

The order department relists material and procures it from the mill. Operating department (the shop), cuts, punches,

* See "Roofs and Bridges," by Merriman and Jacoby, Part III, Chap. IV. See Eng. Record, Vol. XLVIII, pp. 360 et seq., pp. 620 et seq.

rivets, and paints, thus transforming this material into beams, ties, and struts. Inspection department examines them to be sure of their accord with drawings and specifications. When passed, shipping department weighs them, then loads on cars which take pieces to their destination. Here erection department put them into final position. Financial department collects the contract price, pays the men, and subdivides cost.

Art. 35. Plant in General

The site of a plant should be approximately level and must have ample shipping facilities. Freight charges on raw material from mills to plant plus that on finished material from plant to site is to be kept as low as possible. Abundance and cheapness of labor are important.

The framework of the building is usually steel. Any standard roofing or siding may be employed. For the latter, brick or concrete is preferred, but they are expensive and hard to alter.

The grouping of the buildings should be such as to reduce cost of handling to a minimum. It is usual to so arrange the yard that raw material begins at one end and gradually works through to the other, coming out as the finished product. This keeps length of haul down to a minimum. To economize on cost, systems of narrow gage yard tracks should be installed, see Fig. 35. In connection with these, there are numerous bridge and jib cranes, Fig. 36. Together the track system and the cranes do all of the heavy moving and a great deal of the lighter.

Raw material is unloaded by cranes from cars on the siding to stock yard, Art. 36. When shop is ready for fabrication, the cranes load it on the push cars whence it is taken through the shop. At each place where work is to be done, it is lifted off cars and afterwards replaced. Another crane at the other end of the works, loads on cars for shipment.

The principal buildings are:

Offices,	Main Shop,	Forge Shop,
Power Plant,	Machine Shop,	Foundry.
Templet and Pattern Shop,		

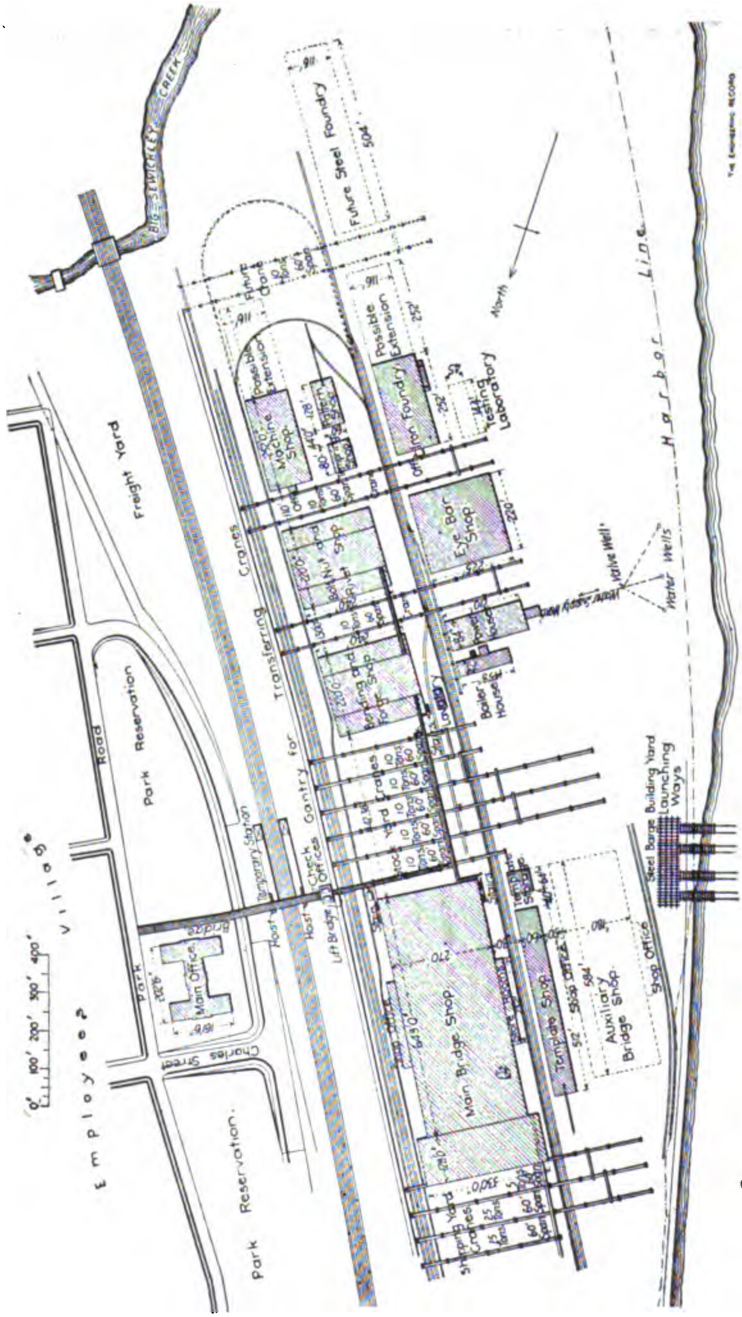

Big Sewickley Creek
Future Steel Foundry
Possible Extension
Machine Shop
Iron Foundry
Testing Laboratory
Freight Yard
Cranes
Eye Bar Shop
Bolt Nut and Rivet Shop
Transferring
Bending and Forge Shop
Power House
Boiler House
Valve Well
Water Wells
Harbor Line
Park Reservation
Road
Gantry for
Check Offices
Temporary Station
Main Office
Park
Charles Street
Employees Village
Main Bridge Shop
Template Shop
Auxiliary Bridge Shop
Shop Office
Steel Barge Building Yard Launching Ways
Shipping Yard Cranes
Stock Yard Cranes
Lift Bridge
Hoist
North
Ohio River
0' 100' 200' 300' 400'

FIG. 35.

The offices are preferably located at entrance, near enough to the works to be convenient, yet far enough away to escape the smoke and noise. A large part of this building, usually the upper floors, will be devoted to drawing rooms. These should be light and well ventilated. Space must be provided, not only for the regular estimating and detailing forces, but also for the erecting and operating departments. The former designs tools and appliances for erection work, Art. 49. The latter performs a similar office for the equipment of the shop.

In the power house are located the necessary boilers and engines. The steam which the former produces may be used:

(*a*) Directly.

(1) To warm buildings. (2) Power. (3) Water supply.

(*b*) Or converted into:

(1) Electricity for light. (3) Pneumatic pressure.
(2) Electricity for power. (4) Hydraulic pressure.

(*a*1) and (*a*3) are outside the scope of this book. (*a*2) is not common. Electric lighting consists of arc lamps for general lighting and incandescent for the individual tools. Electricity for power may be furnished by motor to machines through direct connection or line shaft and belts. Generally speaking, compressed air is used for portable drills, reamers, riveters, chippers, and so forth, also for furnishing draft, for sand blast, and for painting. Hydraulic pressure is employed in forging, upsetting, and riveting—80 lbs. per sq.in. for pneumatic and 800 for hydraulic represent average practice. Occasionally two systems of different pressures, either in air or water may be used.

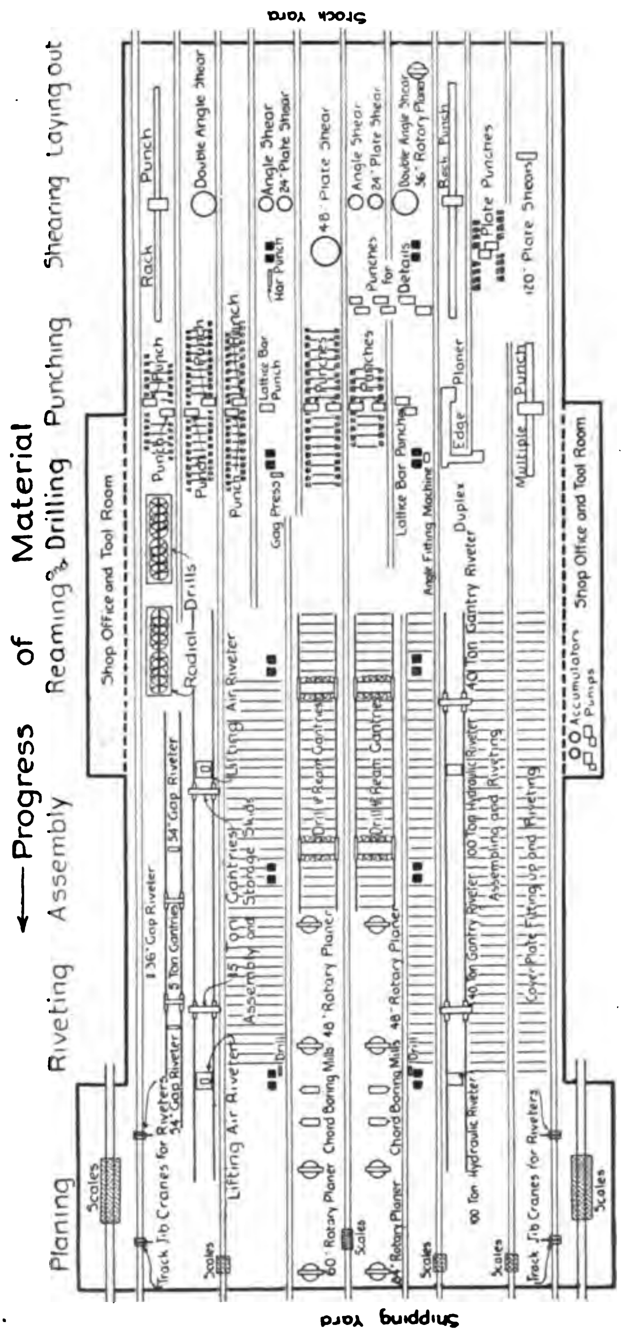
We shall not consider the foundry here as its principles have been taken up in Art. 16.

Art. 36. Stock Yard. (Fig. 36.)

Material is unloaded by cranes from railroad cars and deposited in the stock yards. Size, length, and contract number are marked thereon and it is placed where it can be found when needed. The yard may or may not be under cover. Some

FIG. 36.—Pennsylvania Steel Co.

shelter must, however, be provided for the machines. These are principally straightening rolls. There are two kinds, one for plates and another for angles. Either consists of a number of adjustable rolls between which steel is passed. Other shapes may be straightened by hammering. Shears and cold saws sometimes occur but they are more appropriately described under the head of " Main Shop."



ARRANGEMENT OF MACHINERY
MAIN BRIDGE SHOP
American Bridge Co
Ambridge Pa
Scale 1" = 120

Drawn from blue print loaned
to the author by the
American Bridge Co
Jan 12 1912

H R T

FIG. 37a.

Art. 37. Main Shop. (Fig. 37a and b.)

Here the main operations of fabrication are carried on.

At the end nearest the stock yard, a space is reserved for laying out material, either on the steel itself, or by templet, Art. 40.

Next come the shears. These may be

(1) Beam, (3) Plate, (5) Cross.
(2) Angle, (4) Split,

FIG. 37*b*.—Main Shop, Pennsylvania Steel Co., Harrisburg, Pa.

In shearing, two strong plates are so adjusted, Fig. 37*c*, that they just slip by one another. When cut is made, it must begin at edge, and successively shear off the material. The supports and means for handling different shapes vary.

In the beam shear, a number of different blades are necessary

in order to fit various sizes of I beams and channels. It is not a common tool.

An angle shear is shown in Fig. 37*d*. May be single or double as shown. In the better class of machines, they are made to revolve so that a skew cut may be made without moving piece They should be powerful enough to cut an $8'' \times 8'' \times 1''$ angle.

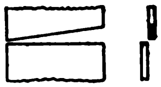


FIG. 37*c*.
Shear Blades.

Fig. 37*e* represents a plate shear. This too may be fixed or revolving. A fully equipped shop will have one machine capable of cutting $120'' \times 1''$ plate.

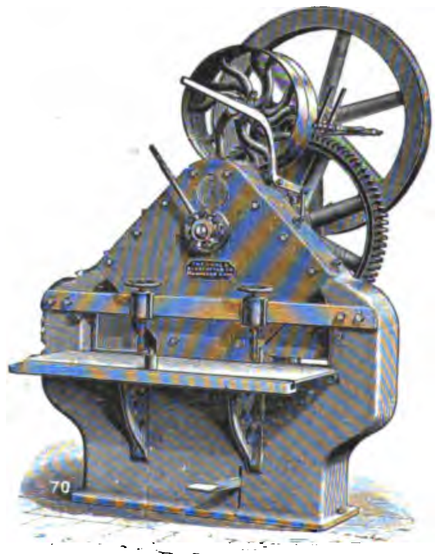


FIG. 37*d*.—Double Angle Shears, Long & Allstatter, Hamilton, Ohio.

There are also small shears as seen in Fig. 37*f*. The blades may be parallel to the axis of the machine (cross shears), or perpendicular thereto (split shears).

Among the accessories, we may name the circular cold saw and a machine for planing the rough edges of sheared plate. Also the coping machine, Fig. 37*g*, which shears off the flanges of I beams and channels; after this has been done,

remainder is a plate and may be easily taken off by an ordinary shear which is a part of the machine. See Fig. 37*g'*.

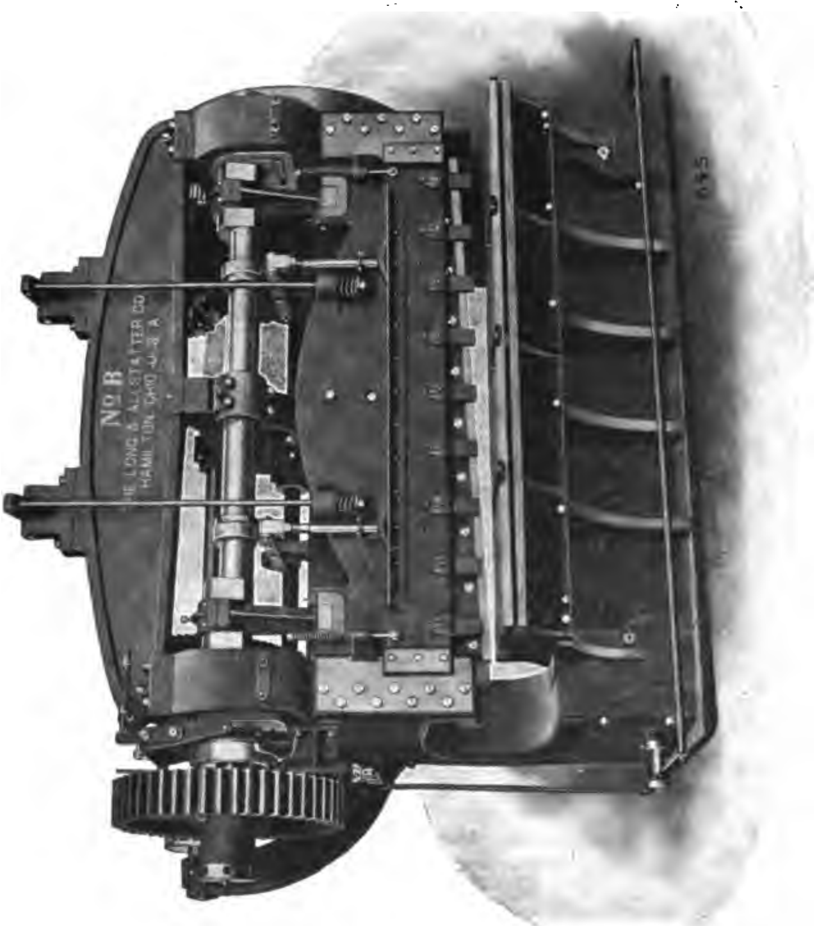


Fig. 37*e*.—Large Gate Shears, Long & Allstatter Co., Hamilton, Ohio.

Next come the punches, which may be classified as follows:

FIG. 37*f*.—Shear, Baird Machinery Co., Pittsburgh, Pa.

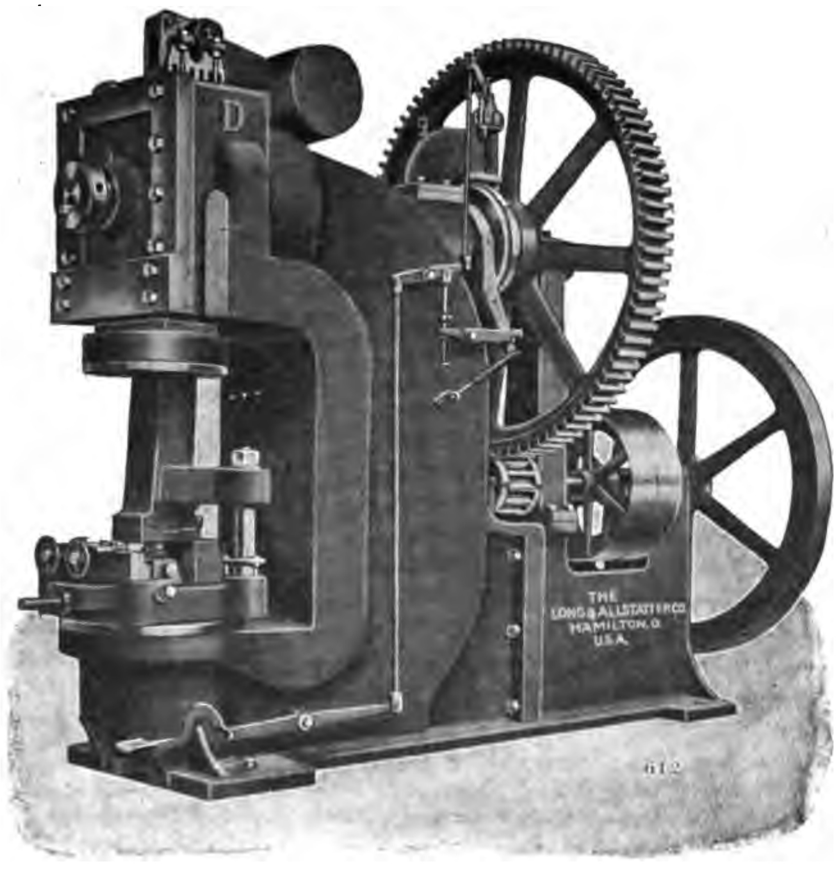


FIG. 37*g*.—Coping Machine, Long & Allstatter Co., Hamilton, Ohio.

(1) Single.	(3) Rack or spacing tables.	(4) Multiple.
(2) Gang.		(5) Special devices.

Essential idea of a punch is a rod of steel passing through

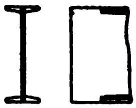


FIG. 37g'.

FIG. 37h.

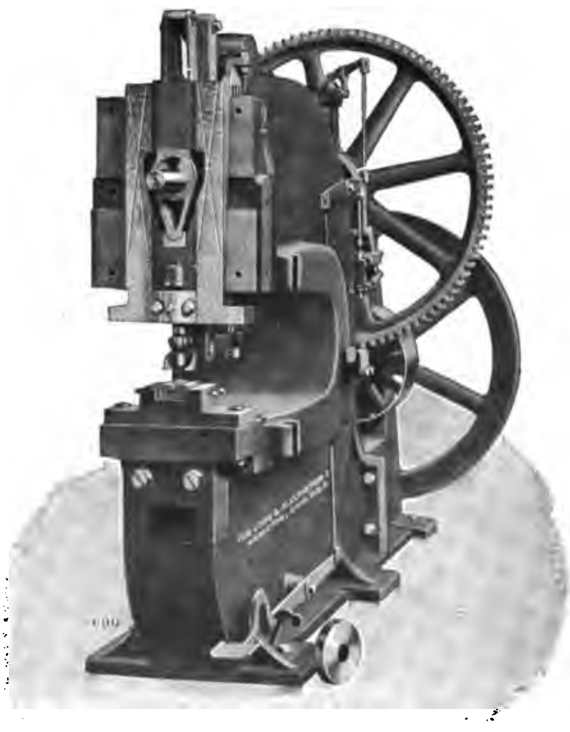


FIG. 37i.—Single Punch, Long & Allstatter Co., Hamilton, Ohio.

a hole just a trifle larger. If now a plate be put between the two, the former forces out an irregularly shaped piece of metal as shown in Fig. 37h.

(1) is shown in Fig. 37*i*. Twice the distance between tool and back of throat is the width of the largest plate it can punch.

(2) is similar to (1) except that there are several punches

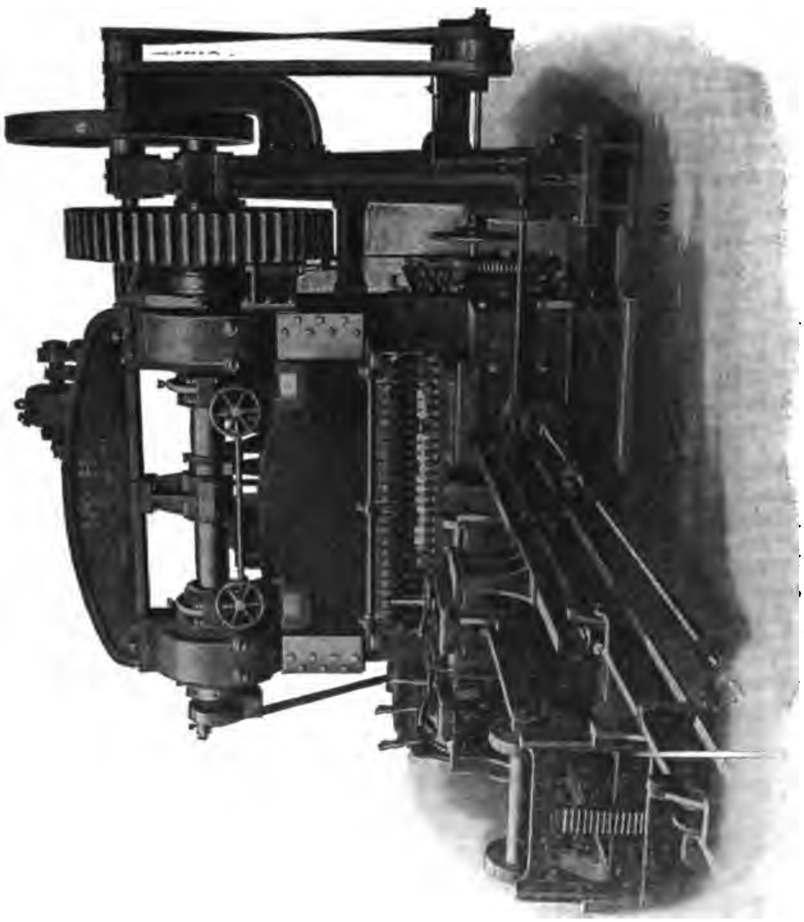


Fig. 37*j*.—Multiple Punch, Long & Allstatter Co., Hamilton, Ohio.

instead of one. It may be used for any standard grouping of holes.

In (3) there are a number of punches in a line, usually four, with a device that enables any combination to be used

at one stroke. The shape which is being punched is fed forward by a mechanically operated table. One type has, at the side of track for table, numerous pieces of steel. A lever in the side of table engages one of these pieces. When released by attendant, it catches on next piece. Of course, these must be carefully set.

(4), Fig. 37*j*, is like (3) except that it is larger and has more

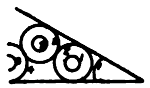


FIG. 37*k*.
Punching Washer Fillers.

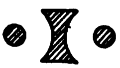


FIG. 37*l*.
Punch for Lattice Bars.

punches. They sometimes have sufficient width for 120″ plate.

Among the special devices, we may mention a machine with two dies of different diameters. This punches out material as shown in Fig. 37*k*. The numerals signify the number of

FIG. 37*m*.—Pneumatic Drill, Chicago Pneumatic Tool Co., Chicago, Ill.

the stroke. Another machine has a punch as shown in Fig. 37*l*. It is used to cut out lattice bars as shown in Fig. 56*x*.

Drills are used for boring holes in metal. Reamers are fluted tools employed to enlarge a hole already formed. Both may be used in the same machine. There are four common types:

(1) Pneumatic drill.
(2) Ordinary drill press.
(3) Radial drill.
(4) Boring mill.

(1), Fig. 37*m*, is a small compressed air engine or turbine, turning a shaft to which is attached the tool. It obtains its air from the supply lines by means of armored hose.

(2) as shown in Fig. 37*n*, is the usual drill press.

FIG. 37*n*.—Ordinary Drill, Baird Machinery Co., Pittsburgh, Pa.

FIG. 37*o*.—Radial Drill.

(3), Fig. 37*o*, is hardly less familiar. It consists of small drills on swinging arms so arranged that they can move back and forth. They are mounted in gangs and their support may be movable.

(4) There are two kinds of boring mills: the horizontal and

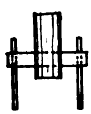


FIG. 37*p*.

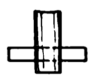


FIG. 37*q*.

Tools for Boring Mill.

the vertical. They are much like drills and can be employed as such. In general, they are used for large holes. The tool for boring eyebars is shown in Fig. 37*p*. One like Fig. 37*q* will enlarge a punched hole.

Methods of driving rivets will be taken up later. There are four types of machines:

(1) Percussion. (3) Pneumatic hydraulic.
(2) Toggle joint. (4) Hydraulic.

A percussion riveter, Fig. 37*r*, is a piece of steel forced back

FIG. 37*r*.—Pneumatic Riveter (without tool), Chicago Pneumatic Tool Co., Chicago, Ill.

and forth like a steam pile driver or a rock drill. This impinges on a stationary die which forms the head of the rivet. Motive power is compressed air from the company mains.

In (2), Fig. 37*s*, an air piston works on a toggle joint, thus

FIG. 37*s*.—Toggle Joint Riveter, Chicago Pneumatic Tool Co., Chicago, Ill.

producing required pressure. Satisfactory adjustment is quite difficult.

(3). Here the air piston acts on a plunger which in turn compresses oil. This drives the piston carrying the riveting die.

(4), Fig. 37*t*. This consists of two jaws, on each end of which is a die. One of these is attached to a powerful hydraulic piston. Air may also be used here.

FIG. 37*t*.—Hydraulic Riveter.

(1) is portable. (4) is usually fixed, while (2) and (3) may be either. Whether it is best to make riveter portable or not depends upon size of piece handled.

Among other machines, we may mention the end milling machine, or rotary planer, Fig. 37*u*, in which a rotating wheel carries in its circumference teeth which remove metal to form a smooth surface. Sometimes there are two planers so made that they can be put a fixed distance apart. Again they are mounted to rotate in order to plane to a bevel.

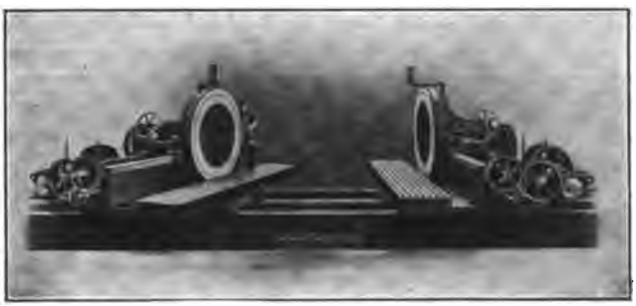


FIG. 37*u*.—Rotary Planer, Niles-Bement-Pond Co., NewYork.

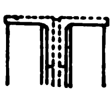


FIG. 37*v*
Stiffeners.

FIG. 37*w*.
Tool for Fitting.

FIG. 37*y*.
Tool for Fitting

Chipping work is done by a pneumatic tool similar to riveter except that a chisel is substituted for the die.

Another machine which may be found in main shop is one for fitting the stiffeners of a plate girder into the flange angles, Fig. 37*v*. The superfluous metal may be removed by a special milling machine with a cutter like Fig. 37*w*, or a shaper with tool like Fig. 37*y*. See next article for a definition of these machines.

Art. 38. Machine Shop

The functions of a machine shop are threefold:

(1) To do mechanical work on structural parts. Such are drilling, boring, planing, turning, and milling. It will be noticed that many of these operations occur in other shops. This is because it saves time and labor when handling heavy pieces to keep tools near by. We find then that the machine

FIG. 38*a*.—Lathe, Baird Machinery Co.

shop takes up this work only when members are light or there is a great deal to be done on them. Among such, we may mention the turning of pins and rollers, the threading of pins, pin-nuts and pilot nuts, the planing of base plates and castings, and so forth.

(2) To do machine work where a part of a structural job. Such, for example, is the machinery in turntables and movable bridges. Another example is the making and repair of erection tools.

(3) To manufacture and maintain such machines and tools in the plant as are not purchased outside.

The idea of machine work is either to cut holes or recesses in metal, for example, a slotted hole, Fig. 61*b*, or a key seat; or

to make a surface exact. Thus a roller must be truly cylindrical, and surfaces which slide must be exact planes.

The principal machines found here are:

(1) Drill presses.	(3) Lathes.	(5) Planers.
(2) Boring machines.	(4) Shapers.	(6) Milling machines.

(1) and (2) have already been explained in preceding article.

Lathes, Fig. 38*a*, have a revolving support and a tool-holder which travels parallel to piece but is made adjustable to bring

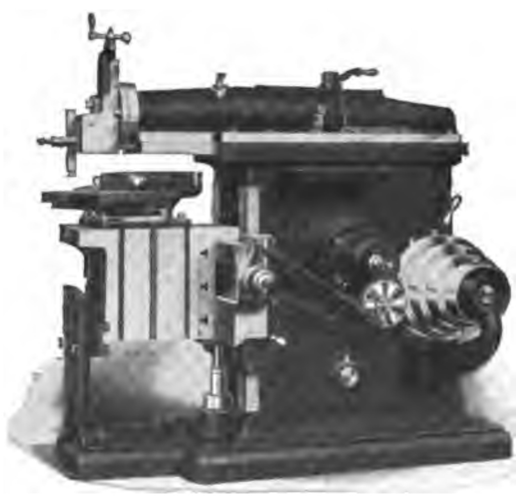


Fig. 38*b*.—24″ Crank Shaper. Queen City Machine Tool Co.

tool nearer to or farther from work. As is obvious, they finish a surface, generated by the revolution of a straight, broken, or curved line about the axis of rotation. They may also be employed to make screw threads.

In shapers, Fig. 38*b*, piece has slow lateral motion, while tool in an adjustable support moves across the work. This will finish plane and cylindrical surfaces generated by a straight, broken, or curved line; its principal use is for plane surfaces.

Planers, Fig. 38*c*, form the same kind of surfaces as shapers. However, the piece moves instead of the tool which has a slow

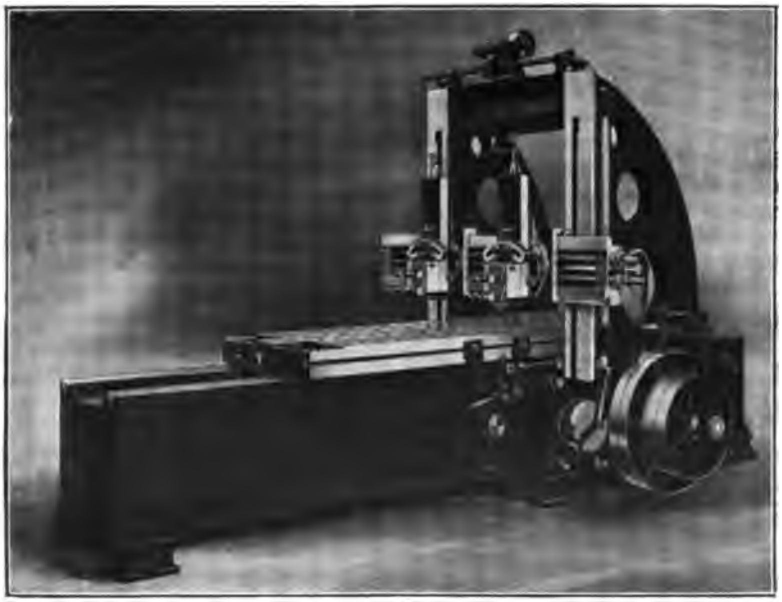


FIG. 38c.—Cincinnati Planer Co.

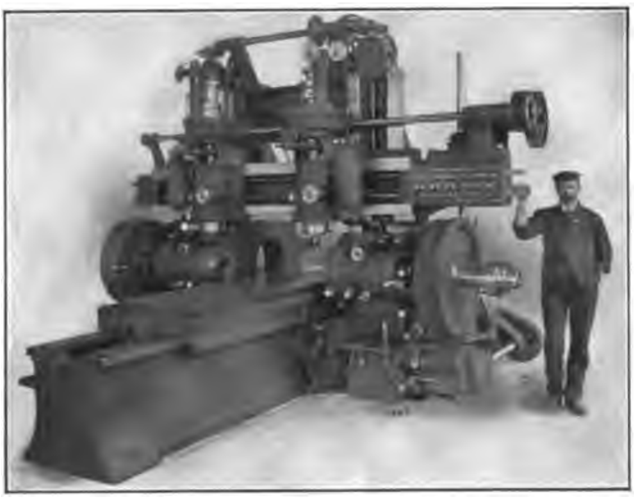


FIG. 38d.—42" Planer Type Milling Machine. Niles-Bement-Pond Co., New York.

lateral motion and is adjustable. In general, planer is employed on large work.

Milling machine, Fig. 38*d*, is one for forming exact surfaces by means of revolving tools or cutters.

An average allowance for plates is 1/16″ for each planing.

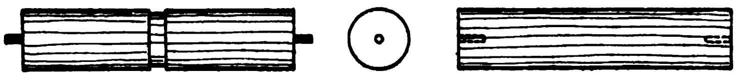


FIG. 38*e*. FIG. 38*f*.

Plain Rollers.

In ordering hot rolled rounds, add $\frac{1}{8}''$ to turned diameter; for forged, add $\frac{1}{4}''$.

As an example of machine work, let us take rollers as used for large expansion bearings. All exterior surfaces except

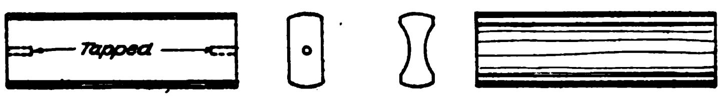


FIG. 38*g*. FIG. 38*h*.

Segmental Rollers.

ends must be finished. For Figs. 38*e* and *f*, rollers are sawn from rounds, and then turned in a lathe. Figs. 38*g* and *h* may be planed and turned from a round, from a rectangular bar, or from a forging of about the same shape.

Art. 39. Forge Shop. (Fig. 39.)

Here we make rivets and bolts, upset eyebars, manufacture miscellaneous forgings, and do general blacksmith work. We find in this shop, rivet- and bolt-making machines into which the heated rod is thrust and from which it emerges in small pieces, cut to proper length, and with one end upset to correct shape. Here are located machines for making nuts and putting thread on bolts.

Where the eyebars are made, we find first the furnaces

in which the raw material is heated, next the upsetting machine which enlarges head to the final shape. Then we have a punch which, while the head is still hot, trims off the irregular projections and punches a hole one inch smaller than finished diameter. There are also straightening machines as already described and annealing furnaces to take out any internal stresses

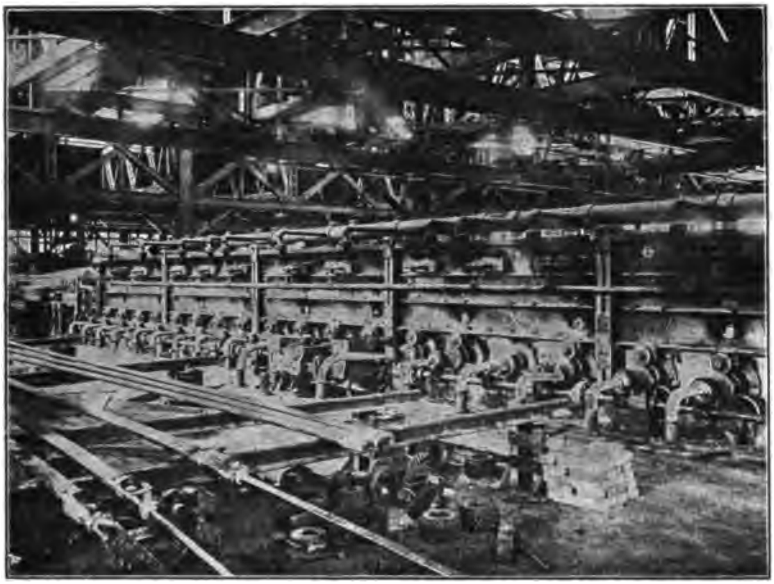


FIG. 39.—Partial View of Forge Shop, American Bridge Co., Ambridge, Pa.

caused by preceding treatment. Near them is located a boring mill, composed of two drills, the distance between which is adjustable.

For the production of miscellaneous forgings and general blacksmith's work, we find the usual equipment. Special mention might be made of the steam hammer and machines for bending steel to a form, " bulldozers."

Art. 40. Templets *

Steel work consists in the main of the rolled shapes, which we have considered in Chap. II, cut to exact dimensions. For the purpose of fastening these different shapes together, holes are placed at frequent intervals. These cuts and holes, made as shown on drawings furnished by the engineering department, are located in four ways:

(1) By laying out on the steel directly.
(2) By templets.
(3) By rack or multiple punch. (See Arts. 37 and 44.)
(4) Special processes.

(1) is the best method where there are only a few holes in a long piece, or where there are only one or two pieces. The mechanic works directly on the steel putting a punch mark wherever a hole is to go while the cuts are indicated by a series of such punches; (2) Templets ("template" is also correct) are pieces of wood designed to be clamped on the steel while the necessary dimensions are transferred from it. Holes about $\frac{1}{2}''$ diameter are bored in the wood opposite those which are to be punched in the steel. Pasteboard or cast iron may also be used as a material.

As an example of (4), we may mention a punch with a special table, carrying a plate to be punched and a pattern plate. When a die attached to machine is placed in any hole in pattern, the punch is directly over same spot in other plate, and accordingly clutch may be thrown in and hole made as usual. Thus this machine saves preparation of templet, and the work of laying out.

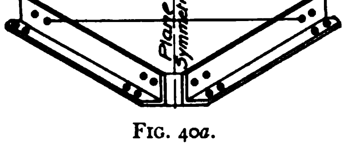


FIG. 40*a*.
Left. Right.

A very common term in technical work is right and left.

If about any plane as an axis, we construct a solid symmetrical with a given solid, the original is called "right" while the other is "left," Fig 40*a*.

Axiom 1. The position of the axis is immaterial, as the resulting left will be the same for all positions.

* Eng. News, Vol. LV, p. 326.

Axiom 2. The left of a body containing a plane of symmetry is the same as its right.

Axiom 3. The left of a composite body is composed of parts each of which is a left of the corresponding parts in the original.

The templet for a small plate is a piece of wood of exactly the same area but not necessarily the same thickness. That for a large plate is usually framed together like a truss with the pieces so placed as to contain the holes.

The templet for an angle is made as shown in Fig. 40*b*. If the angle be right and left, it is constructed like Fig. 40*c*; in case there are holes in one leg only, as in Fig. 40*d*; if there are no holes in a part of one flange, that portion of the templet may be omitted altogether; or the one piece may contain layout for both legs.

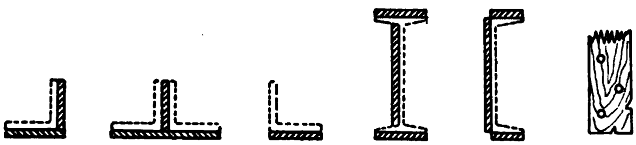


FIG. 40*b*. FIG. 40*c*. FIG. 40*d*. FIG. 40*e*. FIG. 40*f*. FIG. 40*g*.

Templets.

The templet for the top of a channel or I beam is a plain flat board. For the web of the latter it is a board to which are nailed cross pieces containing the holes and just fitting the fillets, Fig. 40*e*. Templets for channel are seen in Fig. 40*f*.

On the templet is marked the size of section, length, diameter of holes, identification mark, and number required.

Templet making is expensive and therefore should be economized as far as possible.

(1) Design as many pieces as possible so that they may go through the rack punch.

(2) If there are two members of the same length but slightly different loads, it is customary to make them alike but strong enough for either. Often a great deal may be saved by bearing this point in mind during the design.

(3) If pieces are not alike, they may be made enough so that a single templet can be used for both. For example

the stresses in the laterals of a bridge decrease toward the center. One templet will do for all panels, however, by using the same sized legs throughout and varying their thickness and by omitting certain rivets; the templet should be marked to show variations. Cuts occurring in one member but not in another may be indicated as in Fig. 40*g*.

The engineering department gives measurements in either of two ways:

(1) Give dimensions sufficient to completely determine everything.

(2) Give general center to center distances, size of all material, number of rivets required, together with maximum and minimum edge distances.

(1) is generally the best method since in case repair or additions are decided on, (2) gives no definite knowledge of the location of the rivets.

A templet shop is usually a well-lighted room with plenty of space for full size layouts. It should contain a full assortment of carpenter's tools, numerous benches, a boring machine, and a knife for cutting off material.

Art. 41. Methods of Cutting Material *

(See Art. 37 for description of machines)

Shapes other than plates, angles, I beams, and channels are usually sawn. However, squares and rounds may be sheared by a special blade. One important point comes in here. It is practically impossible to shear off a distance less than half the thickness of the metal.

Plates are cut either by punching out and chipping, or by one of the shears. The latter should always be used where possible, as the former is very expensive. To see if a given plate can be cut by the shop equipment, draftsmen ought to have a scale drawing, such as is shown in Fig. 41*a*, of the various shears. Note that piece must be reversed to finish shearing. A trial with the plate drawn to the same scale will soon show if it may be cut off in the machine. If impossible, it must be

* Reference for Arts. 41–47, Eng. News, Vol. LV, p. 356.

punched out as shown in Fig. 41*b*, and chipped off, which is very expensive.

Avoid re-entrant cuts, *abc*, Fig. 41*c*. Since the shear cracks material beyond where it has cut, *abc* must either be punched out and chipped or else a hole made at *b* and then *ab* and *bc* can be sheared. This is expensive on account of extra process

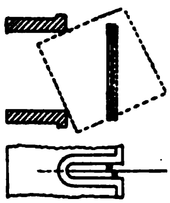


FIG. 41*a*.

Diagram of Gate Shear.

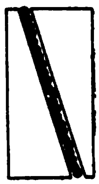


FIG. 41*b*.

Punching Out Plate.

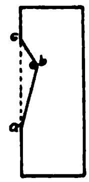


FIG. 41*c*.

Re-entrant Cut.

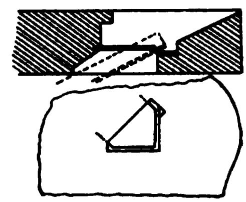


FIG. 41*d*.

Diagram of Angle Shear.

involved. Do not make any angle much less than 90°, as such a corner tends to curl up when shearing.

Angles are usually cut by the angle shears. It saves a great deal of time and often obviates shearing entirely if they have square ends. As will be seen by a study of Fig. 41*d*, there is a limit to the skew at which an angle may enter the

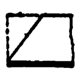


FIG. 41*e*.

Front. Back. Re-entrant.

FIG. 41*f*.

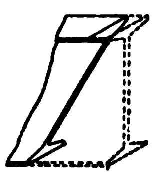


FIG. 41*g*.

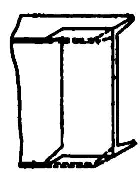


FIG. 41*h*.

Cuts for Various Shapes.

tool. If beyond the limit, angle should be cut off square, and one leg trimmed in plate shear as shown, Fig. 41*e*, two processes instead of one, and that much more expensive.

Fig. 41*f* shows three kinds of skew cuts. The " front cut " is the usual one and angle shears are designed for it. The " back

cut" is more costly but can be done. The re-entrant cut is entirely impracticable.

An I beam or channel may be cut in four ways:

(1) Beam Shears. (2) Coping Machine. (3) Cold Saw. (4) Punching.

(4) is expensive and is done only where the other equipment is lacking. (3) does good work but takes time. (1) and (2) are cheap and good enough. If the beam is to be cut by a plane perpendicular to the web but beveled to the plane of the flange, it may be cold sawn or coped out approximately to required lines as shown in Fig. 41*g*. If cut by a plane perpendicular to flange but beveled to web, it may be cut by cold saw, if latter is large enough or coped out to lines shown in Fig. 41*h*.

Important principles developed here are:

(1) Avoid small cuts.

(2) Do not use re-entrant angles.

(3) Keep cuts to a minimum.

(4) Use square ends unless important reasons determine otherwise.

Art. 42. Methods of Bending

Small rods and thin plates are bent by drawing into position in the assembled structure. In other cases the part must be heated. If much bending of a certain radius is required, two dies of that radius are prepared, one of which is fastened to a plunger. The heated section is placed between them and the stroke of the plunger completes the job. Or the hot shape is hammered or forced by levers into the desired form, the templet being used as a pattern.

A shape may be bent in a plane containing its axis or so as to change the section. As an example of the latter class let us take the connection of two I beams framing together at an oblique angle, Fig. 42*a*. If angle of intersection lies between 85 and 95 degrees, it is customary to use a bent angle. Otherwise a plate is bent the required amount. It is very difficult to make a sharp corner of the latter construction. Generally it will be rounded to a radius of a few inches.

Crimping, Fig. 42*b*, is the offsetting of an angle and is best done by a machine.

Loop rods may be either circular or square in section. The latter gives better contact and is preferred for that reason. They are used only for the purpose of carrying tension between two pins and are commonly made adjustable as explained in Art. 43. To fasten them, the rod is bent back on itself and firmly welded, the distance between pin and weld being equal to

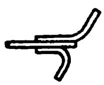


FIG. 42*a*. Bent Connections.

FIG. 42*b*. Crimped Angle.

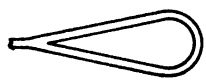


FIG. 42*c*. Single Loop Rod.

FIG. 42*d*. Double Loop Rod.

two or three diameters of the former. This fork is sometimes made double as shown in Fig. 42*d*.

Hangers are short members carrying a direct load. A common construction is to make them like a short non-adjustable rod. The breaking of a hanger supporting a floor beam in a deck railroad bridge near Forest Hills, Mass., on the Boston and Providence R. R. caused a bad wreck, Art. 69, and hangers have not been used in first-class construction since. Indeed, in a well- designed structure, little reliance is placed on a weld joint, its most important allowable duty being for wind stresses.

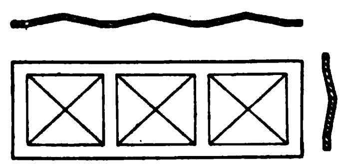


FIG. 42*e*.—Buckle Plates.

Buckle plates are plates stiffened by pressing between large dies into form shown in Fig. 42*e*.

When bent cold within the elastic limit, the length of the neutral axis remains unchanged; above it, the tension side stretches while the compression side does not alter very much. When bent hot, much depends on the manipulation of the metal by the blacksmith.

Forge work injures steel considerably, although subsequent annealing may remove some of this.

A shape can be bent to almost any radius, but the sharper

this becomes, the greater the cost and the loss of strength and the less desirable is the appearance.

We find then that it is difficult to do good work in bending; that it is wasteful and weakening. This conclusion will apply to all hand forgings, hence hand work in the forge shop is to be made a minimum.

Art. 43. Process for Upsetting

This has already been defined in Art. 25. We there spoke of one of the two cases, the other being that of eyebars. The upset for the screw end, whether the shape be round, square, or flat, is made round and of sufficient size after thread is cut to give at least 20% excess of area, this being to allow for loss of strength due to forging, as explained in preceding article. The length of the rod necessary to form this upset may be readily computed if we allow 10% for losses due to heating and trimming.

Adjustable members are sometimes made by cutting in two an ordinary member, upsetting and threading each end so cut, and inserting a turnbuckle, Fig. 43*a*, or sleeve-nut, Fig. 43*b*.

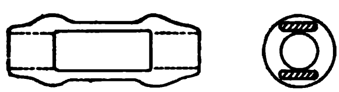


FIG. 43*a*.—Turnbuckle.

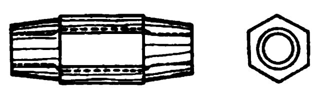


FIG. 43*b*.—Sleeve Nut.

In each, the thread must be right hand in one half and left in the other. The former has the advantage that the position of the ends may be determined by inspection. Eyebars are made by placing a bar in a furnace, heating the end for several feet, and upsetting. Bar is next annealed, that is, heated to a dull cherry red and then allowed to cool very slowly after which they are taken to the straightening rolls. Sometimes a hole somewhat smaller than final is punched while metal is hot and then it is drilled to exact size, or sometimes entire hole is drilled at one operation.

The bar now presents the appearance shown in Fig. 43*c*. The diameter of the pinhole is commonly made 1/50 to 1/32

of an inch larger than size of pin. The thickness of the bar is usually the same throughout, but the head is sometimes thicker than the body of the bar. The net area of the hole should be at least 30% in excess of the area of the bar itself. The head is usually made of arcs of circles, as shown.

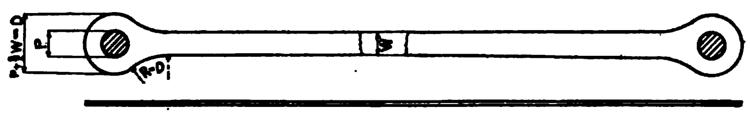


FIG. 43*c*.—Eyebar.

Length of upset; diameter of thread; dimensions of turnbuckles, sleeve-nut, and eyebar heads may differ a little, each company having its standards.

An adjustable member may also be made by upsetting and threading each end and attaching thereto a clevis nut which connects with a pin which in its turn passes through the connection plate. Method of manufacture is as follows: The iron or steel is first piled up as shown in Fig. 43*d*. In the die under the steam hammer, it takes, after being heated, the shape seen in Fig. 43*e*. Reheated and hammered in another die it takes the form of Fig. 43*f*. The smith then bends the

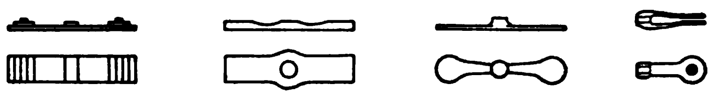


FIG. 43*d*. FIG. 43*e*. FIG. 43*f*. FIG. 43*g*.

Successive Stages in Manufacture of Clevis Nuts.

ends close to the center until they become parallel. The holes for the pin are next drilled and the end is threaded, and the nut appears in its finished form as shown in Fig. 43*g*. Each company has its own standards, the same clevis doing for several sizes of threads or pins, the details being such as to make the nut stronger than the bar.

Art. 44. Methods for Making Holes

(See Art. 37 for machines)

Holes may be either punched or drilled.

A punch is likely to break if metal is thicker than diameter of hole. Do not try partial holes, Fig. 44*a*; it is difficult to secure good work and the punch will not long endure this treatment.

In detailing members where rack or multiple punch will be used, care must be taken to keep rivets in horizontal and vertical lines. There is a minimum allowable distance in each direction, due to the construction of the machine. If spacing is less than that, templets must be used, adding quite a bit

FIG. 44*a*.—Partial Hole.

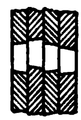


FIG. 44*b*.—Typical Matching of Holes.

to the expense. Rack or multiple punching does not pay where there are but few alike, for small pieces, or for skew work. Other cases should be so arranged that shop can use it if they so elect.

Punching is the cheaper process, but it distorts and injures metal, and holes are likely to match poorly. Further, when several pieces are joined together, the resulting hole, Fig. 44*b*, is irregular and rivet does not thoroughly fill it. To overcome these objections, we may:

(1) Drill from solid, or

(2) Enlarge by reaming.

For the former, there are four methods:

(1*a*) Pneumatic drill.

(1*b*) Drill press.

(1*c*) Radial drills.

(1*d*) Boring machines.

(1*b*) and (1*d*) are much alike, the latter being used for large holes, as already noted. (1*a*) is portable but not as effective

as (1*b*). (1*c*), however, combines the good points of both. Pieces may be laid down underneath the radial drills, while the latter, attached to a small gantry crane, are moved along it, drilling holes as they go.

Now drilling from solid is quite expensive and is seldom done except for cast iron or where metal is almost as thick as diameter of the rivet. The advantages of drilling without its disadvantages may be obtained by sub-punching and reaming, that is, by punching a hole about $\frac{1}{8}$ inch smaller than original diameter of rivet, and then enlarging it to 1/16 inch more. This enlargement may be effected by using either of first three machines mentioned in drilling, with the same or different tools. For reaming also the radial machines are best.

Reaming is done after assembly because it insures a good rivet and takes care of several holes at once. It is often required for field rivets. There are two methods of doing this: by reaming all parts which connect, to a common iron templet; also to put them together at shop, ream, and then matchmark, so the same connection will be made in the field. The latter, while expensive, is obviously preferable and is now quite common.

Pinholes in eyebars or built-up members are punched out and then enlarged in a boring machine. A slotted hole, Fig. 61*b*, may be cut out by a special punch, or two holes may be made at each end, and remainder cut out by shaper.

Important principles are:

(1) Avoid partial holes.

(2) Holes having a diameter less than thickness of material must be drilled to avoid breaking punch.

(3) Different sized holes are a fruitful source of expense and annoyance. See also Art. 47.

Art. 45. Layout and Assembly

Material which requires bending is sent to blacksmith's shop and thence to the room of the layer-out. Straight stuff passes directly thereto. Some pieces will be taken to spacing tables, others punch-marked by measurements on steel itself and perhaps the larger part will be laid off by templet, Art. 40.

Layer-out is supposed to be as economical as possible of mate-

rial. In a large job, involving a great deal of, let us say, 12″×$\frac{3}{8}$″ plates, it will be ordered in about 30 foot lengths. It may be that there are many different lengths and hundreds of pieces to be cut from this. Perhaps there is only one possible way of doing this. The bills of material, Art. 63, give what he is to cut with as little waste as possible. He must also make necessary allowances for fitting and milling. Irregular plates may often be sheared to save a great deal of material. Thus a plate like Fig. 45*a* should be cut out as seen in Fig. 45*b*.

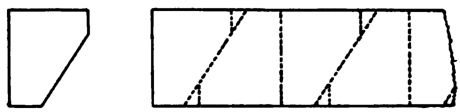


FIG. 45*a*. FIG. 45*b*.
Method of Shearing Skew Plates.

The material is next sheared, Art. 41, then punched or drilled, Art. 44, and passed on to assembly. Surfaces which will afterwards be in contact now receive a coat of paint. The different parts are next "assembled," that is, fitted together and fastened with a sufficient number of bolts to hold firmly while rivets are being driven. Though forbidden by most specifications, holes which do not fit well are persuaded by the use of "drift pins" (pointed pieces of steel), backed up by sledge hammers. A better way and one which is now used more and more, is to sub-punch and ream out after assembly (Art. 44).

It is particularly desirable for this class of work that drawings should be clear, views properly shown, and notes explicit and easily understood.

Art. 46. Fastenings for Steel Work

While there are rivets having different styles of heads as shown in Fig. 46*a*, they are not common in structural work, the button head being the universal type. This is slightly less than a hemisphere. It may be modified as indicated in Art. 47.

Bolts, Art. 25, 7*b*, are also used in steel work. The hexagonal

head is lighter and is preferable on account of its better appearance and lesser clearance required for tightening. Unlike the wood the hole in the steel must be made some larger, and since the bolt is not heated for driving, it does not fill the irregularities as does the rivet. This objection may be overcome by drilling the hole and turning the bolt a few thousandths smaller. It is then called a "turned bolt."

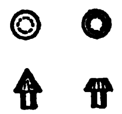


FIG. 46a.
Unusual Rivet Heads.

A tap bolt in steel corresponds to a lag screw in wood. It may have a square or hexagonal head but no nut, and is used to fasten one object to another where it is impracticable to get at the nut end. The piece that carries the screw end is said to be tapped, that is, it has a hole bored in it a little more than the length of the bolt and a female thread is then turned thereon.

A stud bolt is a tap bolt with a thread and nut instead of a head on the outer end. A hook bolt, Fig. 46b, has instead of its head a form bent as shown. It is often used in fastening ties to the beams on which they rest. A U bolt is made as shown in Fig. 46c.

FIG. 46b.
Hook Bolt.

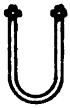


FIG. 46c.
U Bolt.

FIG. 46d.
Ragged Bolt.

FIG. 46e.
Swedged Bolt.

FIG. 46f.
Expansion Bolt.

Foundation bolts are made in various styles, although the plain bolt is about as good as any of them and much more economical; however, it is sometimes threaded; sometimes ragged, Fig. 46d, swedged, Fig. 46e, or is made as shown in Fig. 46f, expansion bolt.

Pins are large specially designed bolts with both ends threaded and a nut placed on each. The usual type of a pin is shown in Fig. 46g. Each company has its standards, but the distance l_2 is usually equal to the metal which it grips, while l_1 and l_3 are each made equal to thickness of the nut plus a

quarter of an inch or a little more. The nut is always hexagonal, and has a long diameter of about 4/3 that of the pin and thickness of $1\frac{1}{4}''$. d_2 is about 3/4 of d_1.

Pins are turned and threaded from rounds of medium steel 1/16 to 3/16″ larger, this amount varying with shop practice and with size of pin.

Sometimes a cast-iron washer about 1″ in thickness and slightly larger than the large diameter of the pin is placed at one end. In that case l_2 and either l_1 or l_3 are each made

FIG. 46g.—Typical Pin.

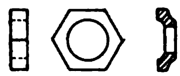


FIG. 46h.—Lomas Nut.

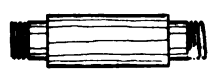


FIG. 46i.—Special Pin.

$\frac{1}{2}''$ longer. The idea of this is to provide for variations in grip from its computed length. A still better method of accomplishing the same purpose is to use the Lomas nut, Fig. 46h, which is about the same size as the usual pin nut except it has a recess of $\frac{1}{2}''$ to $\frac{3}{4}''$ as shown.

Occasionally, for the purpose of providing a smaller hole in a plate, the pin is turned to a lesser diameter usually to that of the thread. This may be done on either or both ends, the latter case being shown on Fig. 46i. Or, at the support, projections beyond the thread may be made in order that jacks be used to lift the bridge.

Suppose in Fig. 46j that it is necessary to keep the outer eyebars 2″ outside of the inner, as shown. If pin and nut as given above be used, the eyebars might move around or rattle. It then becomes necessary to use washer fillers, Fig. 46k.

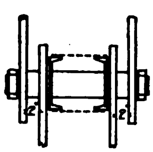


FIG. 46j. Pin Joint.

FIG. 46k.—Washer Filler for Joint.

The plate is usually $\frac{1}{4}''$ thick, and the inside diameter is about 3/16″ more than that of the pin. In this case we would order, "2 Pls. $15''\times\frac{1}{4}''\times1\frac{7}{8}''$," the 15″ being given as the plate width since the same size pins are often used throughout a job, hence, all plate for this purpose will have the same width.

In order to facilitate erection and protect the threads of the pin, a point and a cap must be provided. The point is called a pilot nut and may be either short or long as shown in Figs. 46*l* and *m*. It is cast of iron or steel and has its outside

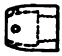
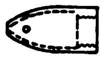



FIG. 46*l*. FIG. 46*m*.
Short. Pilot Nuts. Long.

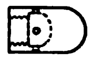


FIG. 46*n*.
Driving Nut.

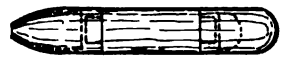


FIG. 46*o*.
Pin Ready for Driving.

diameter equal to that of the pin, while a female thread to fit that on the end of the pin is turned inside. The cap is called a driving nut and is shown in Fig. 46*n*. As in the pilot nut, the driver fits both at the thread and on the outside. The pilot and driving nuts now give the pin the appearance shown in Fig. 46*o*, and render erection quite easy.

Cotter pins are commonly used with clevis nuts. They derive their name from the cotter, a small piece of bent wire as shown in Fig. 46*p*, which is inserted at one end to prevent the pin from coming out. The other end is about $\frac{1}{4}''$ larger in diameter than the body, the whole pin appearing as shown in Fig. 46*q*. l_3 is usually made $\frac{1}{2}''$, l_2 a trifle more than the thickness of metal to be gripped, while l_1 is about 1″. The material is ordered exact to length and of a size of round equaling diameter of head. It is then turned down and a 7/16″ hole for a $\frac{3}{8}''$ cotter pin placed as shown. Fig. 46*q*.

FIG. 46*p*.
Cotter.

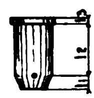


FIG. 46*q*.
Cotter Pin.

Art. 47. Methods for Riveting

(For machines, see Art. 37)

The rivets described in preceding article may be driven by:

(1) Machine riveter. (2) Pneumatic hand riveter.
(3) Hand tools.

The hot rivet is "entered" into hole from one side and firmly held there until tool on other side has upset the head.

At each end is a bar with a nearly hemispherical recess as seen in Fig. 47*a*. Two sides may be cut away or an octagonal bar used. The head mashes up so that $\frac{3}{8}''$ should be employed instead of $\frac{1}{4}''$ in figuring clearances.

In (1), both ends are held by machine, large power is exerted, and it makes an excellent rivet very economically. Hence always design so a machine riveter can be employed as much as possible.

In (2), cylindrical end is upset by the riveter. The other end may be held by a short heavy bar about in line with rivet, by a longer piece called a dolly bar, Fig. 47*b*, used as a lever, or by a pneumatic "holder-on." The latter is a bar with a die in the end which is forced into position against the rivet

FIG. 47*a*.—Rivet Die.

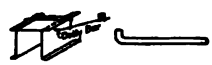


FIG. 47*b*.—Dolly Bar.

and held there by pneumatic pressure. There are several forms of holding bars in order to provide for driving in difficult locations; they may be bent or inclined at an angle to the rivet.

This process makes good rivets but they are not as strong as those driven by machine if proper care is used in adjustment of the latter. The advantages of this process are the portability of the riveter and its adaptability to almost any condition.

In (3), same tools are employed on holding end. The driving is accomplished by a die set in a handle. This die is much like those we have already studied except that it is quite short and head is shaped to receive the blows of the sledges. Resulting rivets are weaker and more expensive than those driven by air and this method is now seldom used; only in small erection work and in locations not accessible to the pneumatic riveter.

In driving the rivet, it is pushed in on the holding side and upset on the driving side. It is best when it can be entered and driven either way. In machine driving, some feet are required in both directions for clearance in line with rivet axis, rather more being necessary on driving side. Measurements

will vary for different machines, and actual details of riveter should be consulted in doubtful cases. For pneumatic riveting, 24″ is desirable on driving side and 12″ on holding, but both can be lessened by special tools, a riveter of this type bringing former down to 12″. This applies to cases where rivet is backed by a pneumatic "holder-on." This is not always necessary, but should be employed if pneumatic process is used in driving long rivets. On the holding side distance can usually be made as small as entering of rivet will permit. The latter applies to hand riveting also, but on the driving side, room must be provided to swing sledges.

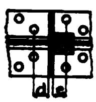


FIG. 47*c*.

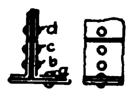


FIG. 47*d*.

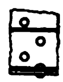


FIG. 47*e*.

Transversely to the axis of the rivet, clearance must be provided for die as already described. In certain locations, parts of machine are to be cleared. Consider, as an example, the vertical rivets in the flanges of plate girders, as seen in Fig. 47*c*. Here the distance *d* for machine driving should be about 2″ and *e* $2\frac{1}{4}$″. In determining clearances, consider only rivet heads that are close by, say within 2 or 3 inches. Those 8 or 10″ away may be ignored entirely. Thus in Fig. 47*d*, to drive *a* after other rivets, consider head *b* but not *d*, and probably not *c*. This interference might be prevented by staggering as in Fig. 47*e*. Following table gives approximate value for clearances:

Diam. Rivet.	Hand Driving.		Machine Driving.	
	Minimum.	Desirable.	Minimum.	Desirable.
$\frac{3}{4}$″	$\frac{7}{8}$″	$1\frac{1}{8}$″	$1\frac{1}{8}$″	$1\frac{1}{4}$″
$\frac{7}{8}$″	1″	$1\frac{1}{4}$″	$1\frac{1}{4}$″	$1\frac{1}{2}$″
1″	$1\frac{1}{8}$″	$1\frac{1}{2}$″	$1\frac{1}{2}$″	$1\frac{3}{4}$″

Clearance required for driving rivets, for erection, or perhaps for other purposes, may require a head somewhat less in height

than full head described in preceding article. It may be, Fig. 47*f*:

FIG. 47*f*.—Special Heads.

1	2	3	4
Flatten to $\frac{3}{8}''$.	Flatten to $\frac{1}{4}''$.	Flatten to $\frac{1}{8}''$.	Countersunk and chipped.

(1) Head flattened to $\frac{3}{8}''$.

(2) Head flattened to $\frac{1}{4}''$, and hole countersunk to obtain the necessary strength.

(3) Head flattened to $\frac{1}{8}''$. Here rivet is pounded down about flat but is not chipped off, care being taken that the head does not exceed $\frac{1}{8}''$.

(4) Head countersunk and chipped flat.

Note well—(1) involves hammering down rivet either by hand or a flat die. (2) and (3), hammering down and boring out of hole (countersinking). (4) means hammering with special die, countersinking, and chipping if special head is on the driving side or a countersunk head rivet and countersinking if on entering side. From the above, it will be seen that countersinking rivets adds largely to the cost.

Points to be kept in mind are:

(1) Be sure to provide sufficient room in all directions around riveter. Be a little liberal in cases where there is close work on more than one side. Provide for entering and holding one way and driving on the other. Proceed similarly for bolts except to substitute clearance for turning nut in place of driving.

(2) If thickness of material gripped by rivet (grip), be more than four diameters, there is likelihood of unsound work. Avoid therefore long rivets.

(3) Countersunk rivets should be reduced to a minimum and if possible eliminated altogether. The additional operations are but a small part of the expense; it is the changing of dies and the care necessary to look after these special rivets.

(4) Different sized rivets in the same piece mean different dies for both punching and riveting. This is costly and a source of much bother. To avoid proceed as follows: For each job,

pick out a certain sized rivet, usually the largest which may be driven. This is about $\frac{3}{4}''$ for small work, $\frac{7}{8}''$ for medium, and $1''$ for heavy construction. Design sections so that one of these may be used throughout. Sometimes it will be cheaper to increase a section and use more metal than to have varying sizes of rivets. When the connections between parts of a structure are few, different sizes may then be used in those parts. In this case, the line of demarcation should be so chosen that the fewest pieces in number and the smallest in size will have to be punched and riveted twice.

FIG. 47g. Buckling of Plate.

(5) If rivet spacing exceeds $6''$ or 16 times the thickness of the thinnest outside plate, there is danger of buckling as shown in Fig. 47g.

(6) Use only as many rivets as are necessary for strength and stiffness. Shop driving alone costs 2¢ apiece, and total expense is not far from 5¢.

Art. 48. Inspection, Painting, and Shipment

Inspection is of two kinds; that controlled by the structural concern and that in the interest of the purchaser. We shall consider inspection of material as outside of this work.

The inspector for the structural company examines each piece and ascertains if outside dimensions and open holes agree with drawings. His principal objects are:

(1) To ensure acceptance of the piece by purchaser.

(2) To correct deviation from plans which would delay completion in case his concern erects bridge.

The work of the purchaser's inspector is much more difficult. Moreover, he is judge between structural company and purchaser as to inspection and fulfillment of contract as shown in the plans and specifications. His field is:

(1) To allow only approved material to be used. Ordinarily this too has been inspected.

(2) To ensure agreement of material with that called for by strain sheet.

(3) To go over joints and ascertain if pieces will fit together

* Reference for Shipping and Erection, Eng. News, Vol. LV, p. 381.

in the field. Not only should measurements be compared with those on the drawings, but the latter should themselves be checked with one another.

(4) To enforce specifications in regard to workmanship. Material must be straight, holes properly punched, and surfaces inaccessible after assembly painted. Inspector should watch milling, boring, planing, and riveting, testing the latter by hammer to be sure that they are tight.

(5) Just before painting, it his duty to make a detailed comparison of each piece with plan, and have any errors corrected.

(6) To inspect painting, weighing, and shipment.

On important jobs there may be an inspector of erection. His duties will be to see that steel work has not been injured in transit, that pieces are put in proper position, and all field rivets are well driven.

Material should be cleaned of rust before painting. On important work, it may be removed by pickling or sand blast; it is sometimes taken off by a coat of gasoline before the paint.

Paint is usually applied by hand with a brush. It may be done by compressed air, which sprays paint on the steel, (also on anything else in the vicinity). It is wasteful, unhealthy, and does not do the work as well but is economical of labor. Pins, pin holes, screw threads, and rollers should be coated with white lead and tallow.

Steel is next weighed and this compared with computed weight, a variation of $2\frac{1}{2}\%$ being allowed. It is then loaded on cars and shipped to its destination.

We shall take up details of preparing pieces for shipment only so far as it concerns designer and draftsman. Pieces less than 40 feet long, 8 feet wide, and 9 feet high may be shipped almost anywhere. If the width or height of a piece exceeds these limits, examine clearance diagram, Art. 50, 5*a*, for roads over which the job must travel. Longer parts may be supported on two cars. Pieces as long as 130 feet may be carried by inserting idler or spacing cars. For such lengths, conditions on curves must be investigated. Plate girders are placed upright on blocks about 6″ high to which they are firmly braced.

The following points should be borne in mind when shipping:

(1) Pieces which, like long plate girders, are difficult to handle, should be shipped so that turning end for end at site will be avoided.

(2) Small parts are likely to be lost in shipment; they may be boxed up, bolted together, or fastened onto a piece with which they connect:

(3) Ship projecting plates or shapes only when necessary to avoid expensive field riveting; otherwise they may suffer injury or interfere with economical shipment.

(4) Freight rates run about as follows: The minimum rate per pound can be obtained when loaded with not less than 30,000 for one car or 45,000 for two. If cars are all occupied, not less than these amounts must be paid for. If only a portion of one car be taken, shipment is made at " less car load " rates which are higher per pound. It is hence sometimes better, in spite of the objectionable field riveting, to ship a small job " knocked down " (in small pieces) and pay pound prices for it rather than full car rates. Remember, however, that the latter is prompter and less likely to result in the loss of a piece, while the former is much easier to team and to handle.

(5) Sometimes purchaser pays freight. Do not forget to guard the interests of your client as far as it is honorable to do so.

Shipping bills are described in the next chapter, Art. 65. They are used as a guide by inspector, shipper, and erector. When everything on this bill has left the shop, its part of the contract is completed. In the next article, we shall take up the work at the final site.

Art. 49. Erection *

Assembling the structure on its site and fastening together is styled the erection. In this article, we shall describe methods for plate girders, viaducts, and simple bridge trusses. Processes peculiar to other types will be considered in chapter dealing with same.

The heavy weights may be handled by block and tackle, jacks, gin-poles, derricks, gallows frames, and travelers.

* See Dubois' "Mechanics of Engineering," Vol. II, Chap XIII.

Hydraulic jacks have a short stroke and are not thoroughly reliable. On account of the latter, blocking must always be used to take up the lift. They may be had with a capacity of 800,000 pounds.

A gin-pole, Fig. 49*a*, is a simple strut of timber or steel, guyed by two or more ropes at the top and supported at the bottom. The hoisting rope runs from a crab near the foot over a sheave at the top and down to the load. It is employed to raise or lower a weight, and is limited as to height of lift by

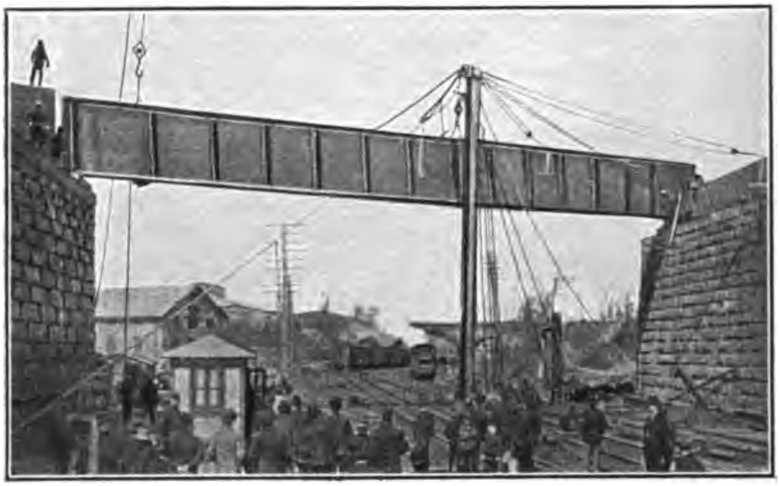


FIG. 49*a*.

upper sheave. Horizontal motion is difficult but can be obtained by manipulating guys with block and tackle. May be moved from place to place by lifting bodily with a derrick, by sliding foot and paying out or pulling in guy lines, or by taking down and setting it up again.

An A frame is much like a gin-pole except that mast is made of two inclined struts braced together and that less guys are needed, one being sufficient. Methods of lifting and moving are also similar.

Steel and wooden derricks, Fig. 49*b*, guyed either by wire ropes or stiff legs are familiar to all. They are moved like gin-

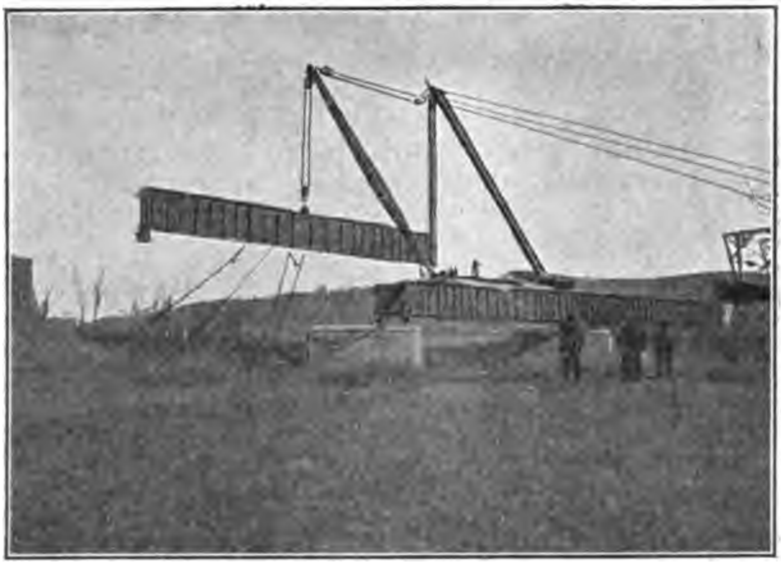


FIG. 49*b* —Derrick Erecting 85-foot Plate Girder on Western Maryland R.R.

FIG. 49*c*.—Derrick Car, American Bridge Co., Ambridge, Pa.

poles. These not only raise or lower the load, but they can place it almost anywhere within a hemisphere whose center is the foot of the mast (the upright piece) and whose radius is the boom (the movable piece). This is likely to be somewhat limited by (*a*) Lack of strength in derrick for certain positions of the load, (*b*) Construction. For example, guys might interfere, (*c*) Objects on the ground.

Derricks mounted on a car, that is, derrick cars, Fig. 49*c*, are very efficient machines. They are commonly provided with a hoisting engine, air compressors, and full sets of tools. The mast is designed to collapse and the boom if a long one is made telescopic; hence they may be shipped from shop to site like an ordinary freight car. It can move slowly under its own power even with load. Its field of action is a semi-cylinder whose axis is a line through the foot of the mast and parallel to the track, and whose radius is the boom. When raising loads at one side, the other may be prevented from uplift by loading an auxiliary boom, Fig. 49*d*, by fastening one wheel to track, or by "outriggers." The latter are beams temporarily bolted transversely to the car with ends supported, loaded, or fastened, to counterbalance the eccentric load.

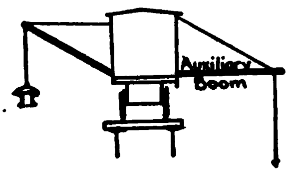


FIG. 49*d*.—Use of Auxiliary Boom.

A gallows frame is shown in Fig. 49*e*. It is guyed at the top and supported at the bottom. A limited amount of motion may be secured by manipulating guys or by varying pulls on the two ropes which support the load. In the main, however, its office is to raise or lower an object.

Travelers are very common in bridge work. There are three types: the ordinary traveler, the cantilever traveler, and the creeper.

The former, Fig. 49*f*, consists of several gallows frames braced together so as to be independent of guying. Sheaves at the top provide necessary number of hoists, while wheels on the bottom allow of longitudinal motion. It can carry a load between any two points within its limits. The inside lines must clear the truss which it is proposed to erect.

FIG. 49e.—Unloading Plate Girders by Gallows Frame.

FIG. 49f.—Ordinary Traveler, American Bridge Co., Ambridge, Pa.

A cantilever traveler, Fig. 49*g*, is one which overhangs. Its essential elements are two projecting trusses well braced together and mounted on wheels, with sheaves at one end and engine and counterbalancing weight at the other.

A creeper is shown in Fig. 49*h*. It is a small derrick, mounted on wheels which run on the top chord of the truss.

There is a great deal of variety in the design of these erection tools, and capacity and measurements differ widely. Space

FIG. 49*g*.—Cantilever Traveler, American Bridge Co., Ambridge, Pa.

does not admit our taking up the subject of design of even the framed structures. In general, these may be made of either wood or steel. Follow conventional methods and use customary allowable values for highway bridges. (Vol. II.) Special care must be taken to get maximum stresses under the varied conditions met in service.

There is even more variety in the situation of the site and in the ingenuity displayed in overcoming obstacles of various sorts. We shall attempt, however, to give only common representative methods.

Three factors are of very great importance:

(*1*) If the site can be reached by rail or navigable waters, it may be termed *accessible*. Otherwise, it must be shipped in small sections.

(*2*) If the traffic must be maintained except for some interruptions of a few hours each, it might be designated *continuous traffic;* such are the difficulties attending construction of this sort that it frequently pays, particularly with highway bridges,

FIG. 49*h*.—Creeper Traveler Erecting Cantilever Bridge at Beaver, Pa. Taken from A. R. Raymer's paper on the "Pittsburgh and Lake Erie Railroad Cantilever Bridge over the Ohio River at Beaver, Pa.," in Vol. LXXIII, Trans. A.S.C.E., opp. p. 156.

to build a temporary structure near by for use during erection of bridge. It may then be treated as a case of *interrupted traffic*.

(*3*) Delivery of material. It may be brought onto tracks running *over* structure which it will replace, or *alongside*, *underneath*, or at *one end* of final location.

ERECTION OF PLATE GIRDERS *

There are five methods, the first three for new structures or interrupted traffic, the fourth and fifth for continuous traffic.

(*1*) Launching, Fig. 49*i*, for inaccessible positions where derricks of sufficient capacity are not available and where material is delivered at one end.

(*2*) Girders may be lifted directly into position (Fig. 49*a*).

(*3*) Tracks, supported by a temporary wooden structure called falsework are built across proposed span. Girders are then suspended, one on either side of flat cars, brought directly over their final position, and carefully lowered.

(*4*) Girders may be brought to site on flat cars and unloaded at or near their final position by overhead hoists supported by falsework.

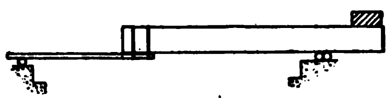


FIG. 49*i*.—Erecting a Plate Girder by Launching.

(*5*) Girders may be unloaded at one side and shoved or lifted into place as old bridge is removed, travel being temporarily suspended.

Up to the capacity of shipping facilities and erection tools, entire structure is preferably riveted up complete. The girders themselves can be shipped in parts, but this is very rare.

The girders may be placed a little more than their proper distance apart, the intermediate pieces put in position, and then the former moved up to fit. A much better way is to make a design in which all intermediate pieces may be "swung in" with girders in final position.

ERECTION OF VIADUCTS †

These have seldom been built to replace old structures. Material is commonly unloaded at one end and there reloaded

* Engineering Record, Vol. LIX, p. 494 et seq.

† Ibid, Vol. LXI, p. 429.

onto contractor's cars, which deliver it to erector's gang. The latter generally employ a derrick car or a cantilever traveler. Beginning at one abutment, they erect in order: the first bent, the first span, second bent, longitudinal bracing, second span, third bent and so on. Or the first tower, the first span, the second span, second tower, and so forth.

ERECTION OF TRUSS BRIDGES

Small trusses may be shipped complete or riveted up at the site and handled like plate girders.

There are four methods of erecting larger bridges:

(*1*) Falsework, Fig. 49*j*. This is the usual way. The falsework is composed of wooden trestle bents. Posts and caps are about 12″×12″ with 3″×10″ bracing, resting upon piles, or, in favorable ground, mud sills. On top of these bents are supported the traveler and the blocking on which the bridge is placed. One proceeding is to begin at or near center, erect panels there on both sides, connecting up with lateral bracing. From there trusses are erected towards fixed end. Traveler is next brought back to center and erection carried on towards free end. Another method is to erect floor system, fasten trusses to floorbeams, in order previously given. After the latter are complete, blocking is removed and bridge settles onto its shoes.

Other three methods are employed where falsework would be impracticable on account of depth or rapid movement of stream.

(*2*) Cantilever. This case is shown in Fig. 49*k*, where the two shore spans would be erected by falsework. The center span would be built as seen in the picture. Toggle joints are provided at *a* and wedges at *b*. Trusses are built a little high and are dropped into position to connect by means of toggles and wedges. One disadvantage of this method is the extra material sometimes needed to carry the erection stresses.

(*3*) In end launching, the bridge is erected on shore. When finished, it is pushed forward on rollers, the projecting end being sustained by a float.

(*4*) In floating, the bridge is built on falsework carried by scows sunk in the water. Latter is pumped out and structure

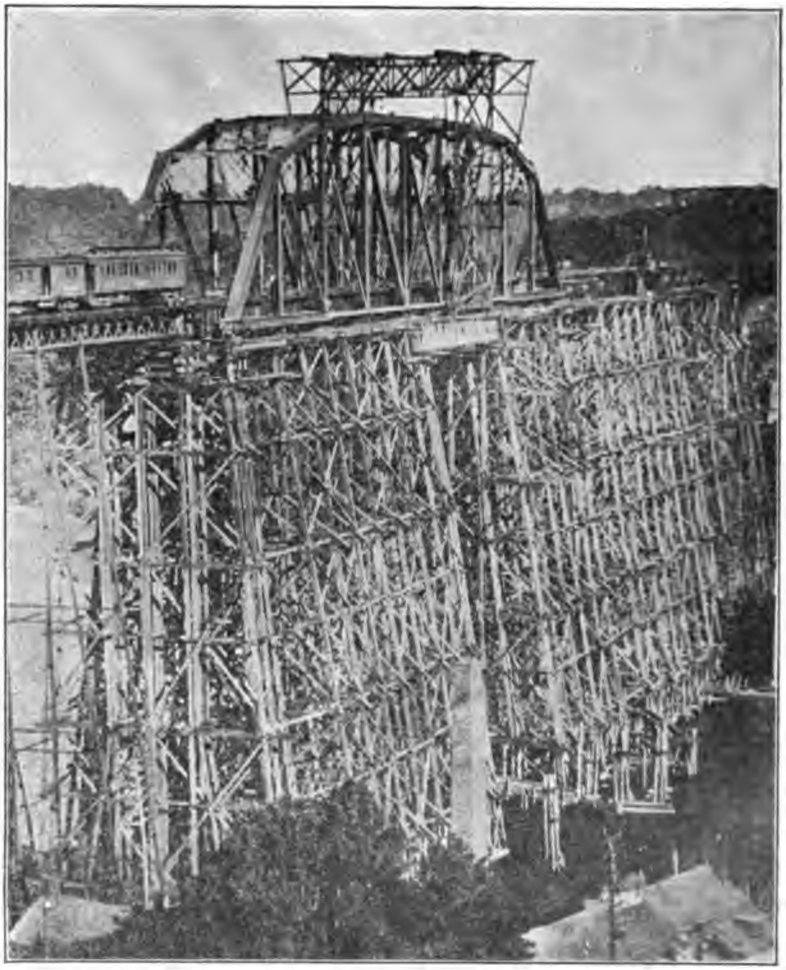


FIG. 49j.—Falsework Supporting Bridge during Erection, American Bridge Co., Ambridge, Pa.

is towed to a position just over the final one, and valves in the bottom of scows are opened.

Pins are fitted with pilot and driving nuts (Art. 46) and driven by means of a wooden maul or a heavy suspended timber.

When assembled, bolts are placed in the rivet holes. Generally but a fraction of the open holes are so filled. Enough, however, must be inserted to carry the maximum stresses which will occur before riveting.

Rivets may be driven by hand or a pneumatic hammer (Art. 37). The former is now confined to very small jobs or places not accessible to the latter. Field riveting is much more expensive than shop, costing 5 to 15 cents apiece, or even more. It varies a great deal with circumstances.

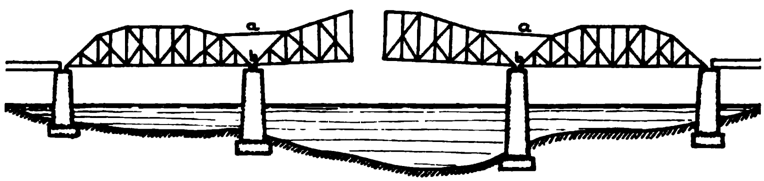


FIG. 49*k*.—Erecting a Truss by Cantilever Method.

The designer should always consider erection and have at least one simple and safe method in mind. Take the plans of the apparatus which will be used and see how every piece will be placed in position. Bear in mind the following principles:

(*a*) Make field riveting a minimum. It is much more expensive than shop yet not as strong.

(*b*) However, this should not make pieces too large or too heavy to be readily handled or shipped.

(*c*) Avoid, if possible, groups of a few rivets in inaccessible locations. It may often cost more to build the platform on which the workmen stand than it does to drive the rivets.

(*d*) Consider how each bolt, pin, and field rivet will be entered and fastened. For an example of difficult driving, see inside field rivets in Fig. 49*l*.

(*e*) Avoid entering joints, Fig. 49*m*. This might be obviated by attaching one or both angles to projecting webs.

(*f*) Allow ample clearances where it will do no harm. Do

not attempt to get closer to an interference than one-half inch unless it will look bad or weaken the structure.

(*g*) Where a horizontal member frames into a vertical surface, for example, connection of stringer and floorbeam, an angle on which to rest the former is a great help.

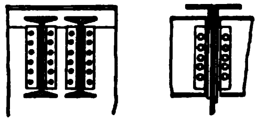


FIG. 49*l*.
Interior Field Rivets are Hard to Drive.

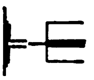


FIG. 49*m*
Entering Joint.

(*h*) Consider how anchor bolts will be placed. The best method is to set them in holes drilled after erection. See that enough room is allowed for drill and hammering same.

(*i*) Avoid members very much alike but not exactly so; a misplacement may be very expensive.

CHAPTER V

THE ENGINEERING DEPARTMENT

Art. 50. Specifications *

THE clauses which define a contract are called the Specifications. In its completest sense, the word covers the necessary legal forms, statements of amounts and limits of work, permissible materials, and rules of procedure. It is mainly in the latter that we are interested. These are used not only as a part of the agreement, but as a guide to contractor's engineers. They may specify simply the finished structure, leaving designer and builder free to exercise their judgment; or they may cover the minutest details and processes. An intermediate course is better. Specifications should be complete, concise, and clear. Useless directions add to the cost, while meager allow inferior work.

As an example, we will give a set of specifications for railroad bridges. This will follow somewhat closely present (1912) average practice. Notes of occasional differences and other comments will be enclosed in brackets.

SAMPLE SPECIFICATIONS FOR RAILROAD BRIDGES

(a) Description

(*1*) PREFERRED TYPES	Up to 30 feet. Rolled I beams. 30 to 100 feet. Plate girders. 100 to 175 feet. Riveted Trusses. Above 175 feet. Pin-connected trusses. Deck bridges shall be used where conditions permit.

* See Cooper's 1906 Specifications for Railway Bridges; Ostrup's Standard Specifications.

(2)
PREFERRED DEPTH

Depth shall not be less than following amounts:
Plate girders, one-tenth span.
Trusses, one-sixth span.

(3)
STRINGER SPACING

Stringer spacing shall be 6 feet 6 inches center to center, except for curves or some other special reason.

(4)
SPACING OF GIRDERS AND TRUSSES

Center to center spacing shall be as follows:
Deck girders, one-twelfth span but not less than 6 feet 6 inches.
Through girders to suit clearances.
Trusses, not less than one-twelfth span.

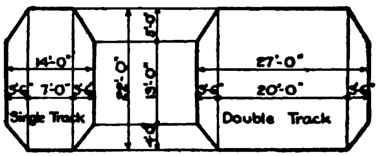


FIG. 50*a*. FIG. 50*b*.
Typical Clearance Diagrams
Single Track. Double Track.

(5)
CLEARANCE DIAGRAMS

Bottom line in Figs. 50*a* and *b* represents base of rail. Widths to be increased to give the necessary clearance on curves.

(6)
TIES AND GUARD TIMBERS

Ties shall be of white oak or yellow pine eight inches wide and spaced as near six inches apart in the clear as is practical. Make depth one-tenth the span but never less than eight inches. Minimum notch over supports, one-half inch. Fasten every third tie to stringer or girder by $\frac{3}{4}''$ hook bolts. Guard timbers to be eight inches wide and six deep notched to 4″ over ties and fastened by $\frac{3}{4}''$ bolts to every third tie.

(*b*) *Loads*

(*1*) DEAD LOADS

Dead loads shall be computed in all cases. Allow 125 lbs. per ft. per track for rails. Timber to be considered as green and estimated as given in Art. 2. Assume ballast to weigh 110 lbs. per cu.ft.

(*2*) OWN WEIGHT

Stresses due to own weight must be allowed for in the design.

[Impact, which is often included in specifications, is covered by formula for unit stresses.]

(*3*) LIVE LOAD

The live load per track shall be taken as Cooper's E 50, as given in Fig. 50*c*. Weights are in thousands of pounds.

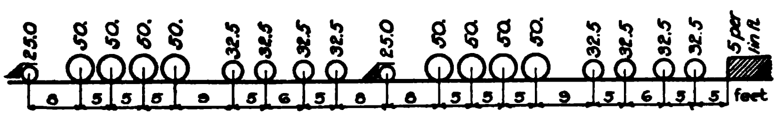


FIG. 50*c*.—Loading Cooper's E 50.

(*4*) CENTRIFUGAL FORCE

Centrifugal force, F, shall be assumed to act at a point 6 feet above base of rail. Let D = degree of curvature, W = live load on bridge, v = velocity in feet per second.

(Assume a speed in miles per hour of $60-3D$).

$F=Wv^2/gr$. Substituting:

$v=(60-3D)5280/3600$ feet per sec.

$g=32.2$ ft. per sec. per sec.

$r=(5760/D)$ in feet.

We may derive the following formula:

$F=.042DW(1-0.1D+0.0025D^2)$.

(*5*) TRACTIVE FORCE

A force equal to one-fifth the live load on the bridge shall be considered to act horizontally along base of rail in either direction.

The wind shall be taken as 30 lbs. per sq.

(6) WIND — ft. acting on structure and train or 50 lbs. per sq.ft. on former alone. Consider all trusses but only one plate girder to receive this pressure.

(c) *Allowable Unit Stresses in lbs. per sq.in.*

Let us take C to represent the safe unit stress for quiescent tension equals 9000 for soft steel, 10,000 for medium steel, and 15,000 for 3 per cent nickel steel. Let

$$M = 1 + \tfrac{1}{2}\frac{\text{minimum stress}}{\text{maximum stress}},$$

$$R = \text{maximum slenderness ratio}$$

$$= \frac{\text{unsupported length}}{\text{least radius of gyration}}.$$

(1) ALLOWABLE UNIT STRESSES

Tension on net section,	CM
Compression on gross section,	$CM(1 - .006R)$
Shear on shop rivets, pins, and gross section of webs,	$2CM/3$.
Shear on field rivets and bolts,	$CM/2$.
Bearing on shop rivets and pins,	$4CM/3$.
Bearing on field rivets and bolts,	CM.
Bending on pins,	$1.5\ CM$
Other flexural stresses,	CM.

(2) COMBINATIONS — The fraction minimum/maximum shall be considered as negative if there is a reversal of stress. Members are to be designed for either (*a*) 80% of maximum stresses due to dead, live, centrifugal, tractive, and wind, or (*b*) the first three. The larger of these two values must be employed.

(3) OTHER ALLOWABLE VALUES — For wood, see Art. 6. Allowable stress per lineal inch on medium steel rollers is 300 times diameter in inches. For bearing on masonry, 300 pounds per sq.in.

(*d*) *Design*

(*1*) Accessibility and Drainage

Sections and details shall be accessible for cleaning, painting, and inspection, and shall not retain water.

(*2*) Minimum Thickness

Except for fillers and lacing bars, minimum allowable thickness of metal is $\frac{3}{8}$ inch.

(*3*) Grip

The grip of the rivets shall not exceed 5 times its diameter (Art. 47).

(*4*) Section Design

Avoid large sectional areas which receive their stresses indirectly [e.g., cover plate on top chord, Fig. 50*d*, should be kept as thin as conditions permit. This is because compression distributes itself unequally, giving the heaviest unit stresses in the webs, *W*, and the least in plate, *P*.]

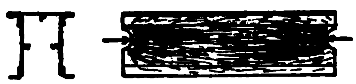


Fig. 50*d*.—Distribution of Stress in a Built-up Section.

(*5*) Angle Connections

If angles be connected by one leg only, its area will be considered as that leg. This clause shall not operate, however, to reduce the radius of gyration.

(*6*) Net Area at Pins

Riveted tension members shall have a net area at pin equal to five-fourths that in body of piece.

(*7*) Net Section

In deducting rivet holes, diameter is to be taken as one-eighth inch greater than nominal diameter of rivet. In staggered spacing, use actual area along the zigzag line if it will be less than that in one plane.

(*8*) Counters

Counters shall be designed for 25% additional live load and a 25% increase in allowable stresses.

(9) MAXIMUM LENGTH COMPRESSION MEMBERS — Maximum slenderness ratio for members carrying wind or tractive forces only, 125; for other pieces, 100. If slenderness ratio for any part of a compound column exceeds that of column as a whole, the ratio for that part shall be used. The two compression chords of truss or plate girder bridges shall be similarly treated as a large column with wind bracing for lacing. [Clause *4a* is intended to eliminate the possibility of lower stresses by this method.]

(10) TRANSVERSE LOADS — Bending stresses due to own weight in compression members may be neutralized by eccentric arrangement of joints (Art. 56). Other transverse loads are preferably avoided but must be allowed for if they occur.

(11) TOP FLANGES PLATE GIRDERS — Top flanges of beams and plate girder must have their allowable stress reduced by the compression formula if unsupported for a distance greater than 15 times width of cover plate.

(12) TOP FLANGE RIVETS — Rivets in the top flanges of deck plate girders and stringers shall be computed for resultant shear and vertical load. The heaviest wheel of the loading is considered as distributed over three feet.

(13) COMPUTATION OF PLATE GIRDERS — In computing plate girders, web shall be designed to carry its share of the bending moment. Compression and tension flanges shall be made alike, and not less than one-third of flange area shall be in angles and side plates.

Stiffeners shall be in pairs. At bearings and points of concentrated loading, they shall have sufficient capacity to carry the load as a column. If thickness of web is

(14) STIFFENERS — more than one-fiftieth of the depth, other stiffeners may be omitted. If not, they shall be spaced one-half to one-third depth apart at ends and depth apart at the middle. At intermediate points, use a proportionate spacing.

(15) BRACING — Complete upper, lower, and sway lateral systems shall be provided where possible; under other circumstances as in through structures, bracing will be designed to be of the necessary strength and as efficient as practicable. Solid floor shall be considered as a lateral system in its own plane. Struts located at or near shoes shall be capable of resisting temperature stresses.

(e) Details

(1) RIVET SPACING — Rivets may be $\frac{3}{4}$, $\frac{7}{8}$ or 1 inch diameter, d. Let t equal thickness of thinnest outside plate, then:

Max. edge distance $= 8t$,
" spacing $= 16t$, but not more than 6 inches.
Min. edge distance $= 1.5d$,
" spacing $= 3d$.

When staggered, spacing is the shortest distance center to center of rivets. Above rules for maximum do not apply to two angles riveted together.

(2) FIELD RIVETS — Number of field rivets shall be kept as low as possible. (Art. 49.)

(3) LATTICED COMPRESSION MEMBERS — A latticed compression member shall be figured for a uniform transverse load equal to 1/30 strength of member as a short strut. (Art. 56.) It shall have as near ends as practical, batten plates not shorter than greatest width of member, and not

thinner than 1/50 of transverse distance between rivets. Longitudinal spacing on batten plates shall not exceed $4d$.

(4) RIVETED COMPRESSION MEMBERS — The transverse distance between rows of rivets in plate shall not exceed $40t$; longitudinally, the space will not be more than $4d$ for a distance from ends equal to twice the greatest width of the member.

(5) SPLICES — All joints must be fully spliced except the milled ends of short well-braced columns. For the latter use two rows of rivets on each side of joint on all four faces.

(6) RIGID MEMBERS — Verticals which carry tension shall be designed as stiff members. In riveted trusses, tension members must be battened or latticed. Connection of floorbeam and trusses must be rigid.

(7) CAMBER — To provide a camber for trusses, make top chord longer than bottom by $\frac{1}{8}$ inch in every 10 feet.

(8) SHOES — Provision for an expansion of $\frac{1}{8}$ inch in 10 feet shall be made. Spans over 75 feet must have hinged shoes, fixed at one end and rollers at the other. The diameter of these rollers in inches shall exceed by 3 the span in feet divided by 100. Rollers and pins shall be made of medium steel.

(*f*) *Workmanship*

(1) Workmanship shall be first-class in every respect.

(2) PUNCHING AND REAMING — Where material is thicker than diameter of rivet, hole must be drilled from the solid. All other holes for shop and field rivets shall be punched $\frac{1}{8}$ inch smaller than nomi-

nal diameter and reamed to 1/16 inch larger after assembly.

(3) RIVETS — All rivets must be tight, completely fill the hole, and have full round concentric heads.

(4) TURNED BOLTS — When replacing rivets, bolts must be turned to fit.

(5) SHEARED EDGES — Sheared edges of medium steel plate over $\frac{5}{8}$ inch thick shall be planed.

(6) STIFFENERS — Stiffeners and their fillers must be fitted to flange angles.

(7) UPSET ENDS — Welds are forbidden. Eyebars and upset ends shall be annealed. Strength of either of the last two must exceed that of body of bar.

(8) PIN HOLES — Holes shall be placed in center of member unless otherwise shown; clearance of pin in hole shall be 1/50 inch if diameter be less than 5 inches; 1/32 inch, if more. Distance between holes shall not vary more than one-twenty-thousandth from true length.

(9) ADJUSTABLE MEMBERS — Adjustable members shall be avoided where possible. When necessary, use turn-buckles.

(g) *Painting and Erection*

(1) SHOP PAINT — All steel before leaving shop shall be cleaned of rust and given one coat of paint. Surfaces which will be in contact afterwards, must be painted before assembly. Pins, pin holes, screw threads and rollers shall be coated with white lead and tallow before shipping.

(2) FIELD PAINT Parts not accessible after erection shall receive one coat at shop and one at site before erection. Other parts shall receive two coats in their final position. Painting shall not be done in wet or freezing weather.

(3) PINS Pilot and driving nuts must be used in driving pins.

[For the sake of conciseness, we have omitted paints and other materials. In specifying them, use, if possible, some standard specifications.]

Art. 51. Problem of Design

The site of the proposed structure should first be surveyed and a map prepared. The object of this is:

(1) To locate the structure in the most economical position.

(2) To enable its details to be specified in advance.

(3) To determine quantities and estimate cost.

Property lines, buildings, contours, soundings, and borings should be taken. The latter ought to extend to rock, and if there is doubt as to its integrity, should be examined by the diamond drill, which withdraws a core for examination. Consider every possible location, a few rough measurements often showing the impracticability of some designs. It may be necessary to place the structure at a particular spot, but it is even then better to make the survey for reasons (2) and (3).

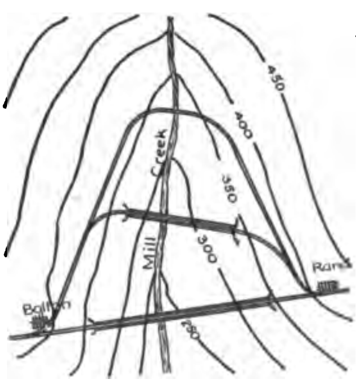


FIG. 51.—Alternative Routes for Railroad Location.

To indicate the necessity for thorough information, let us take the case shown in Fig. 51. The problem is to locate a railroad between the two towns and design the necessary struc-

tures. The cost of operating would be least if alinement and grade were straight from one town to the other. But first cost would probably be lessened by:

(a_1) Going farther up the valley and using a shorter bridge but a longer line.

(a_2) Cutting deeper into the hill at the towns, and so lessening depth in valley.

(a_3) Lowering grade in valley.

Various modifications and combinations of these should be worked out.

(b_1) Consider now that alinement and grade for railroad are chosen—shall we use a viaduct or a bridge with abutments or fill all the way across, leaving only a culvert? In the first two cases, where will it be economical to stop the fill and begin the viaduct or bridge?

(b_2) Suppose location and ends of fill to be settled, shall we use timber, steel, stone, concrete, reinforced concrete, or combinations of these?

(b_3) After material is selected, what shall be the spans? Shall they be uniform, making superstructures alike, or shall they increase as valley becomes deeper? We might use one very long span or many very short ones, but obviously the former would result in an exceedingly expensive bridge, while the latter would give many abutments and piers, thus increasing cost.

(b_4) Closely allied to this is the type of bridge. Considering now only a steel bridge, shall it be arch, suspension, cantilever, or simple truss? Shall we use the through or the deck structure?

(b_5) Type having been determined upon, what should be the depth and what the panel length?

(b_6) Coming down now to the design of the members of the truss and taking the individual pieces, what combinations of structural shapes will carry the stresses economically and make easy and efficient joints?

(b_7) Even in the details of the latter, there are various types each having its own advantages.

In this example, we have indicated only a few of the questions which are interwoven with the problem of design. Some, it will be noticed, are a little outside the province of the structural engineer. We will answer them all in the same way:

CHOOSE THE MOST ECONOMICAL DESIGN WHICH GIVES THE REQUIRED DEGREE OF SAFETY.

For elements of cost, see Art. 27. (a_1), (a_2), and (a_3) belong to Railroad Engineering. In (b_1), speaking roughly, we would stop fill at a depth where cost of fill per lineal foot became greater than cost of bridge. But it must be remembered that maintenance charges are much lower for a properly built fill and its sinking fund would be zero. These operate therefore to increase depth where fill stops.

(b_2) Timber is cheaper in its first cost but maintenance and renewal charges are higher. Properly built masonry bridges have neither maintenance nor sinking fund charges, but their first cost is high, they cannot be used for long spans, and they are ill suited to foundations on compressible soils such as are quite common. We shall confine ourselves to steel structures in the remainder of this treatise.

The rest of our work will be taken up in discussing the remaining points for different structures. Special reference may be made to Art. 52 for (b_3) and (b_5); Arts. 54, 55, and 56, (b_6); Arts. 58, 59, 60, (b_7).

Very often considerations other than economical ones will govern. Such, for instance, are the architectural appearance or legal difficulties. All possible alternatives, however, must be very carefully investigated. Do not make arbitrary decisions but prepare estimates of cost until certain that the most desirable scheme has been found.

Plans are better if completed before construction is begun, although it is true that many changes will have to be made as the work progresses. Estimates of cost can then be finished with much more ease and economy. These do not have that uncertainty which is likely to mean high bids from responsible contractors.

To estimate the cost, quantities are computed from the preliminary plans. Since these will probably be changed more or less in actual construction, rules which are only approximately correct are often employed. Each quantity multiplied by its estimated price gives the total amount to which something like ten per cent should be added for contractor's profit.

Often the designing and estimating for the steel work is

done by the fabricating company. However, if there is no competition, the purchaser will pay dearly for advice obtained in that way. If competitive designs are requested, the buyer really pays for them all. Structural steel is let either by the pound or the lump sum, the latter signifying a fixed amount for the whole job. In the former case, the contractor may try to use as heavy material as he can; in the latter, as light as possible, that is, he tries to " skin " the bridge. To prevent this, plans and specifications should be thorough and explicit, simply allowing the necessary latitude for varying shop practice.

Best method is then as follows: The general design and sizes of all material are determined by purchaser's engineer. Plans and specifications are next prepared and sent to prospective bidders. The latter take off the quantities in the estimating room and make their own estimate of cost, usually on the basis of structure ready for traffic. This with an allowance for profit is submitted as a bid. The lowest responsible bidder is then given the job.

Art. 52. Economical Relations

Given a structure of a certain type, there are relations between measurements which produce the most economical design.

(*1*) As an example, let us take the case of a plate girder of a given span and loading. The weight of the web and its fittings is about constant per square foot. Letting h represent depth, we may then say:

$$\text{Total weight of web} = C_1 h.$$

Using approximate method of computation, Art. 54, the area of the flange and hence its weight will vary inversely as the depth, or

$$\text{Total weight of both flanges} = C_2/h,$$

$$\text{Total weight of girder} = C_1 h + C_2/h. = \text{W}$$

Placing first derivative equal to zero to obtain value of h which renders weight a minimum:

$$\delta W/\delta h = C_1 - C_2/h^2 = 0,$$

or

$$C_1 h = C_2/h.$$

Hence, make depth of girder such that weight of both flanges equals that of the web and its fittings.

The above proof assumes, as often happens, that minimum thickness of web suffices. Where it does not, $W = C_1 + C_2/h$. Hence, for the latter case, increase depth as much as practical or until minimum thickness of web is reached.

(2) Let a river crossing have x spans of length L/x and suppose the foundations to be the same throughout its length. The cost of the steelwork for the floor will be constant and we will call it F. That of trusses will vary as square of span and we will represent it by $CxL^2/x^2 = CL^2/x$. The cost of each pier will be about the same whatever the span and we will call each P and total amount Px.

The entire structure then has a cost:

$$E = F + CL^2/x + Px,$$

$$\delta E/\delta x = -CL^2/x^2 + P = 0 \quad \text{or} \quad Px = CL^2/x.$$

Hence cost of piers equals that for trusses in an economical structure.

(3) For the comparison of trusses, we may find the sum of the products of maximum stress in each member by its length. That truss for which this sum is a minimum is the most economical. However, this method does not take into account one very important fact: that for the same stress, steel compression members are a great deal more expensive than tension members. The following example will explain not only how this may be allowed for but also show method of applying calculus.

Everything in kips and inches.

Allowable unit stresses:

$$\text{Tension} = 8.0,$$

$$\text{Compression} = 8.0\left(1 - \frac{1}{160}\frac{L}{\rho}\right),$$

if $\rho = \dfrac{A}{4}$, then

$$\text{Compression} = 8.0\left(1 - \frac{1}{40}\frac{L}{A}\right).$$

Let S = allowable unit stress in tension or pure compression; T = total stress; length of member $= L$.

Then volume in tension $= TL/S$,

Then volume in compression $= AL = TL/S\left(1-\frac{L}{40A}\right)$

or $$SAL = TL + SL^2/40$$

or $$A = T/S + L/40,$$

$$V = AL = TL/S + L^2/40.$$

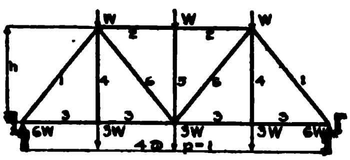


FIG. 52.—Economical Height, Pratt Truss.

Member.	Total Stress, T	Length, L	Volume of 1, V	Number.	Total Volume.
1	$-6W\frac{(p^2+h^2)^{\frac{1}{2}}}{h}$	$(p^2+h^2)^{\frac{1}{2}}$	$\frac{6W}{S}\frac{p^2+h^2}{h}+\frac{p^2+h^2}{40}$	2	$\frac{12W}{S}\frac{p^2+h^2}{h}+\frac{p^2+h^2}{20}$
2	$-8W\frac{p}{h}$	p	$\frac{8W}{Sh}p^2+\frac{p^2}{40}$	2	$\frac{16W}{Sh}p^2+\frac{p^2}{20}$
3	$+6W\frac{p}{h}$	p	$\frac{6Wp^2}{Sh}$	4	$24\frac{Wp^2}{Sh}$
4	$+3W$	h	$\frac{3Wh}{S}$	2	$\frac{6Wh}{S}$
5	$-W$	h	$\frac{Wh}{S}+\frac{h^2}{40}$	1	$\frac{W}{S}h+\frac{h^2}{40}$
6	$+2W\frac{(p^2+h^2)^{\frac{1}{2}}}{h}$	$(p^2+h^2)^{\frac{1}{2}}$	$\frac{2W}{Sh}(p^2+h^2)$	2	$\frac{4W}{Sh}(p^2+h^2)$
			Total..........	.....	$\frac{56Wp^2}{Sh}+23\frac{Wh}{S}+\frac{p^2}{10}+\frac{3h^2}{40}$

$$\frac{\delta V}{\delta h} = -\frac{56Wp^2}{Sh^2}+\frac{23W}{S}+\frac{3h}{20}=0,$$

or $$3Sh^3+460Wh^2=1120Wp^2.$$

Here $W=10$ dead. $S=8.0$. $p=240$.

Substituting, $h=247=20'-7''$.

For doubtful or irregular cases where above methods cannot be used, make several designs and estimate quantities and costs. The greater the magnitude of the work, the more need for time. Do not spend a large amount of money upon a small or typical structure but follow current practice. A saving of \$50 in material at a cost of \$100 in drawing room is poor engineering.

There are very many practical points interwoven with this question. For example, the most economical length of an I beam span will be one where a standard depth of the minimum weight is just loaded to its full capacity. Again in a viaduct of varying depth, span should change to correspond as indicated in (2) of this article. However, spans thus determined might be too long to be easily erected, and a great deal of time might be saved in designing, detailing, and fabricating, if they were made alike or perhaps in two or three different lengths.

Art. 53. Estimating

While, strictly speaking, this includes only the approximate determination of weights and cost, the estimating room of a structural company has following functions to perform:

(*1*) To determine what kind of a structure shall be used in a given location and what its general dimensions shall be. (Arts. 51 and 52).

(2) To specify loads and unit stresses. (Art. 50).

(3) To compute total stresses in various parts of the structure.

(*4*) To design these members. (Arts. 54, 55, 56).

(*5*) To outline in a rough way the details.

(*6*) To estimate weight and cost of the different items.

(*1*), (*2*), and (*3*) ought to be done by purchaser's engineer, and (*4*) and (*5*) are often so handled.

An estimator must hence be thoroughly posted on both

theoretical and practical work. His is the highest position in the engineering department which does not require the handling of men.

(*6*) is best done by writing a rough bill of material and thus getting weight and cost. This, carefully made, is accurate and detailer can keep his work close to the estimated figures. However, it is usually done in a hurry, and there is seldom time for anything but the roughest sort of drawings. Hence the estimator tends to forget certain parts and therefore underestimates.

Time is often too short even for the above. Then the section may be considered as running from joint to joint and either a fixed length, a fixed amount, or a percentage then added. This varies a great deal with so many conditions that we cannot attempt to give values here. It should be figured out for typical members or abstracted from similar cases. Weight of rivet heads are often added as a percentage and this too is quite changeable.

An even more approximate method is to use formulæ expressing some relation between weight and the dimensions of the structure. Such is:

$$W=0.5as(1+0.15s),$$

as already given for wooden trusses in Art. 28. Or we may estimate from diagrams plotted from actual weights of completed structures.

Allowance should be made for waste and for possible variation of weight in rolling. Where work is to be let by the pound price, only relative amounts are necessary and accuracy is not so essential. The cost on each different class of raw material is determined by consulting price card of steel company. For other items, a number of very important points arise:

(*a*) Are members unlike one another or may they be made the same? The latter lowers cost everywhere and especially in pattern, templet, and drawing room.

(*b*) Is the structure skew or square? The former involves a great deal of expensive blacksmithing, and notably increases labor in detailing and fabricating.

(*c*) Are the specifications unusually strict? Many desirable features are costly, for example, sub-punching and reaming.

(*d*) Are erection conditions favorable? Is the site handy to the railroad? Will unusual equipment be required? Is there danger of the structure being carried away by floods or ice?

(*e*) Is enough time allowed for economical work?

Very rough average cost for erected steel is about as follows—1912.

	Cents per lb.
Material	1.20
Engineering	0.15
Templet	0.08
Shop	0.80
Erection	0.60
General expenses	0.17
Transportation	Variable

Or 3.0 cents per pound plus transportation.

The pieces of which a structure may be composed are beams, tension members, and columns.

Art. 54. Design of Beams

Beams may be made of angles, I beams, channels, T beams zee bars, rails, trough sections, or built-up members.

Plates are sometimes used for flooring. As an example of their use, let it be required to find allowable span of $\frac{3}{8}$ inch medium steel plate when subjected to a load of 500 lbs. per sq. ft., using specifications of Art. 50. This is a uniformly loaded beam. Considering a strip one foot wide: Total load $=(500+15)/12=43$ lbs. per lin.in.

$$M=Sbh^2/6, \quad S=10{,}000. \quad b=12, \quad h=\tfrac{3}{8}.$$

$$=10{,}000\cdot 12\cdot 9/(64\cdot 6)=2810 \text{ lb.-in.}$$

$$l=(8M/w)^{\frac{1}{2}}=(8\cdot 2810/43)^{\frac{1}{2}}=23 \text{ inches allowable span.}$$

Although probably continuous, American practice is to regard it as a simple span. The minimum is so small that it is considered as zero in formula for allowable stress.

Angles, zee bars, and channels are not symmetrical about center line. There is danger that the load may be applied

eccentrically and thus cause an injurious twisting, Fig. 54*a*. Nevertheless, they are used quite a bit in situations where their form renders them more convenient. As an example, let it be required to design a soft steel angle to carry a load of 300 lbs. per lin.ft. for a span of 8 feet. Allowable stress in flexure, 16,000; shear, 12,000, both in pounds per sq.in.

$$\text{Maximum moment} = Wl/8 = 2400 \cdot 96/8 = 28{,}800 \text{ in.-lbs.}$$

$$I/c = M/S = 28{,}800/16{,}000 = 1.8.$$

Use $5'' \times 3'' \times 5/16''$ weighing 8.2 lbs. per lin.ft. with long leg parallel to the load. Testing for shear:

$$S = V\Sigma az/bI^* = 1200 \cdot 1.66 \cdot 3.32 \cdot 0.31/(0.31 \cdot 6.26) = 1050 \text{ lbs.}$$

per sq.in., max. shearing stress. O. K.

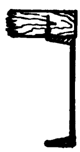


FIG. 54*a*.—Application of Load to a Channel Used as a Beam.

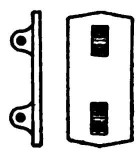


FIG. 54*b*. Separator.

A symmetrical section could be made out of this by fastening together two angles whose joint capacity would be 600 lbs. per lin.ft. However, an I-beam would be more economical as the angles would have to be riveted together and the former would weigh less.

Thus to design an I-beam equivalent to the two angles whose combined weight is 16.4 lbs. per lin.ft., we look for one with a section modulus equal to or greater than 3.6. We might use a $4''$ at 10.5 lbs. or a $5''$ at 9.75 lbs. Assuming as usual that shear is uniformly distributed over the web, the shearing unit stresses on gross area are $2400/(4 \cdot 0.41)$ or $2400/(5 \cdot 0.21) =$ 1460 or 2280 lbs. per sq.in. They are both O.K. even allowing for possible rivet holes. It will be noted that while the $5''$ beam has less weight it is not quite $\frac{1}{4}''$ thick. It is likely, therefore,

* Merriman's "Mechanics of Materials," Art. 108.

to lack stiffness and has less resistance to corrosion. Either would effect a marked saving in weight.

Two I-beams may be used as a single beam. They should be united by bolts which pass through separators, Fig. 54*b*. The idea is to stiffen the top flanges and make the two act together. Two I-beams are not as economical as one, but they are sometimes used where head room is limited or where one is not sufficient. It should be remembered that the Bethlehem Steel Co. is now rolling some sections deeper than 24 inches and also special sections of less depth which have larger resisting moments than the standard. (Art. 21.) As a measure of this increased capacity, the following table shows safe loads for spans of twenty feet; allowable fiber stress, 16,000 lbs. per sq.in.:

Standard Sections.					Bethlehem Sections.			
No.	Depth.	Weight per Foot. lbs.	Total Weight per Foot. lbs.	Capacity. lbs.	No.	Depth.	Weight per Foot. lbs.	Capacity. lbs.
3	24″	80	240	278,400	1	30″	200	325,000
2	24″	80	160	185,600	1	24″	140	187,000
2	18″	55	110	94,000	1	18″	92	95,000

The common type of a built-up beam is shown in Fig. 54*c*. The web is usually kept away from the back of angles $\frac{1}{8}''$ to $\frac{1}{4}''$. Sometimes it is made flush and chipped or milled off. The idea is to prevent pockets for the accumulation of dirt and moisture. This makes a good but expensive job. Occasionally the web is allowed to project. Such may be the case in the supporting beams (stringers) for a railroad bridge where it makes the necessary notching of the ties (called dapping) easy. This can be done only where there is no cover plate. The rectangular plates shown dotted may be added to increase its strength. Ordinarily, they do not extend the entire length of the girder, but are cut off to correspond with variations of the bending moment.

The angles may have equal or unequal legs; in the latter case, the longer leg is placed horizontally since it is more effective in this way.

For heavier beams, there are a number of arrangements, Figs. 54*d* and *e* being representative types. Where a heavy load is to be borne and the depth is limited, a box girder, Fig. 54*f*, may be employed. Sometimes more than two webs are used.

Considering now built-up sections, there are two methods of computation, the exact and the approximate. The former assumes that the stress varies as the distance from the neutral axis, the web as well as the flange bearing its share of the moment. The latter supposes entire moment to be carried by the flange.

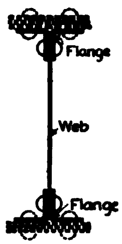


FIG. 54*c*.

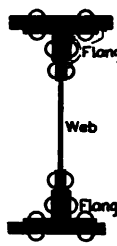


FIG. 54*d*.

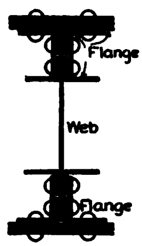


FIG. 54*e*.

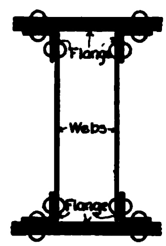


FIG. 54*f*.

Typical Beam Sections.

(*a*) *Design of Web*

In either event, shear is considered to be uniformly distributed over the web. This is approximately true. We have the formula for shearing unit stress in lbs. per sq.in.

$$S = V\,\Sigma az/bI.$$

where V = total shear on vertical section in lbs.;

Σaz = statical moment about center of gravity of part of section above point where shearing unit stress is desired—computed in inches;

I = moment of inertia about center of gravity of entire section in inches;

b = breadth at same point in inches.

Now as we pass up from center of a beam shaped like a plate girder, there is no change in V, I, or b, and there is little change in Σaz until flange is reached when stress drops off very

rapidly as shown in Fig. 54*g*. That is, as the shear over the web is uniform and as there is little on flange, we assume entire amount to be uniformly distributed over web.

(*b*) *Exact Method of Finding Flange Area*

Taking up now the exact method, we assume a composition of the flange which we estimate to be sufficient. We then compute I and c as though it were solid except for rivet holes, and determine unit stress in lbs. per sq.in. at outside fiber of beam from the formula:

$$S = Mc/I,$$

where M = bending moment at section in in. lbs.;

c = max. distance in inches from center of gravity to outside fiber;

I = moment of inertia about center of gravity in inches.

If S comes too high or too low, we revise and recompute.

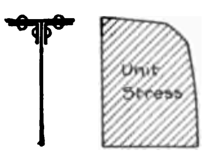


FIG. 54*g*.—Distribution of Shear in a Half Beam.

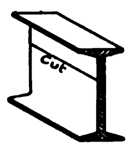


FIG. 54*h*.—Part of a Solid Beam.

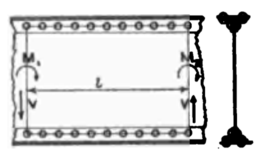


FIG. 54*i*.—Part of a Built Beam.

(*c*) *Exact Method of Determining Rivet Spacing*

In the formula, $S = V\Sigma az/bI$, if we omit b, it represents the total shear per lin.in. That is, if we should cut an I beam as shown in Fig. 54*h* and then load it, we would find longitudinal motion along plane of cut. To prevent this we would need a force for every lineal inch, equal to $V\Sigma az/I$. This force, it will be noted by referring to the proof, is equal to the difference per lin.in. of total stresses in cut off part. Similarly, in a built-up beam, the tendency of some of the shapes to shear off is given by the same formula. Thus, in Fig. 54*i*, enough rivets must be passed through the angles to safely carry this force.

If, as often happens, there is a vertical load on top or bottom of the girder, the vertical stress per lineal inch must be combined with the horizontal to obtain resultant. It is usual for ease of fabrication to make both flanges and their rivet spacing alike, hence only spacing on loaded chord need be computed.

(*d*) *Approximate Method of Finding Flange Area*

In the approximate method, the moment of inertia is twice the net area of the flange, a, times the square of half the distance, h, between centers of gravity of flanges. This neglects moment of inertia of web and flanges about their own centers of gravity as already explained. Moreover, it assumes that distance to most strained fiber is equal to $\frac{1}{2}h$. Substituting in flexure formula already given:

$$S=\frac{Mc}{I}=\frac{M\frac{1}{2}h}{2a\frac{1}{4}h^2}=M/ah \quad \text{or} \quad a=M/Sh.$$

(*e*) *Approximate Method of Determining Rivet Spacing*

To investigate the rivet spacing by this method, let a built-up beam, Fig. 54*i*, be subjected to the bending moments M_1 and M_2 at distances l apart. Let V be the average shear for that interval. As already shown the flange rivets have to carry the change in stress between the two sections. Now the stress at left section is M_1/h; at right, M_2/h; their difference is:

$$(M_2-M_1)/h.$$

But $$M_2=M_1+Vl,$$

hence, $$S=\frac{M_1+Vl-M_1}{h}=\frac{Vl}{h}.$$

Shear per lin.in. $=s=S/l=V/h$. This must be combined with vertical shear as before.

(*f*) *Intermediate Method of Finding Flange Area*

An intermediate method of finding flange area is to consider that the web carries its share of the moment and that the center

to center of gravity of flanges, the depth of web, and twice the distance from neutral axis to most strained fiber, are all alike. Then,

$$S = Mc/I = M\tfrac{1}{2}h \Big/ \left(2a\cdot\tfrac{1}{4}h^2 + \frac{th^3}{12}\right) = M \Big/ \left(ah + \frac{th^2}{6}\right),$$

or
$$a = \frac{M}{Sh} - \frac{th}{6},$$

where t is the thickness of the web.

To allow for holes in the web, $\frac{1}{6}$ is made $\frac{1}{8}$. That is, in this method, we use approximate formula except that we deduct $\frac{1}{8}$ gross area of web from flange area required.

(g) *Example Showing Approximate Method of Computation*

Let us now design in medium steel a beam of 30 feet span to carry a dead load of 700 lbs. and a live of 5000 both per lineal foot. Use approximate method and stresses as given in Art. 50. Rivets, $\frac{7}{8}$ inch dia. Estimating dead load of beam at 300 lbs. per lin.ft.:

$$1 + \frac{\text{Min.}}{2\ \text{Max.}} = 1 + \frac{1000}{2\cdot 6000} = 1.08.$$

To obtain economical depth, we will assume a $\frac{3}{8}''$ web and multiply its area by 1.65 to allow for stiffeners and fillers. For flanges, we will use for comparison net area required.

$$M = 6000\cdot 30\cdot 30\cdot 12/8 = 8{,}100{,}000 \text{ in. lbs.}$$

Depth, Web.	Area Web. Sq. In.	Area × 1.65. Sq. In.	Effective Depth.	M/Sh Flange Area Required. Sq. In.	Area × 2. Sq. In.	Total Area. Sq. In.
30″	11.25	18.6	28″	26.8	53.6	72.2
36	13.50	22.3	34	22.0	44.0	66.3
42	15.75	26.0	40	18.7	37.4	63.4
48	18.00	29.7	46	16.3	32.6	62.3
54	20.25	33.4	52	14.4	28.8	62.2
60	22.50	37.1	58	12.9	25.8	62.9

The theoretic minimum occurs between 48″ and 54″ and verifies proof in Art. 52. We will take the former. Its weight per foot adding 20% for rivet heads and bracing, will be:

$$62.3 \cdot 1.20 \cdot 3.4 = 254 \text{ lbs.}$$

Proceeding with design:

Gross area required in shear $= 6000 \cdot 30/2 \cdot 7200 = 12.5$ sq.in. Necessary thickness of web $= 12.5/48 = 0.26''$. Use $\frac{3}{8}''$, the minimum allowable. Net area required in each flange is 16.3 sq.in. Using a section like Fig. 54*c*, and deducting one 1″ dia. rivet hole in each angle and two in each plate,

2 Ls, $6'' \times 6'' \times \frac{1}{2}''$.	Gross area = 11.50.	Net = 10.50 sq.in.
1 Pl. $14 \times \frac{1}{2}''$.	7.00	6.00
Total,	18.50	16.50

Back to back of flange angles will be made 48.5″. Distance between centers of gravity is then:

$$48.5 - 2(10.50 \cdot 1.68 - 6.00 \cdot 0.25)/16.5 = 46.5''.$$

	End.	6′ out.	12′ out.
Vertical force on rivets per lin. in.	500	500	500
Maximum shear in pounds	90,000	57,000	30,000
Horizontal shear in pounds per lin.in. $= V/h$	1,960	1,240	650
Resultant shear per lin.in	2,020	1,340	820
Spacing, rivet value 4710 lbs	2.34″	3.50″	5.75″

Values of rivet, $\frac{7}{8} \cdot \frac{3}{8} \cdot 14{,}400 = 4710$ lbs. in bearing
$2\pi(7/16)^2 \cdot 7{,}200 = 8640$ lbs. shear

Computation for vertical rivets is similar. Considering increments of stress as proportional to areas, which is approximately true, shear per lin.in. between plate and angle equals:

$$\frac{\text{area of cover plates}}{\text{area entire flange}} \cdot \frac{V}{h}.$$

The vertical force is, of course, zero.

Distance from end	0	6	12 feet
V/h	1960	1240	650 lbs. per lin.in.
Area cover pls./area flge. $= \dfrac{6.00}{16.50} = .364$			
$.364\dfrac{V}{h}$	710	450	230 lbs. per lin.in.
Value of two rivets in single shear 8640 lbs.			
8640/shear per lin.in. = spacing	12.2	19.2	37.5 ins.

(*h*) *Same Problem, — Exact Method*

Now let us try the same problem by the exact method, assuming equal depth. Computation for the web will remain unchanged. We next test the flanges, assuming 7/16″ metal therein:

$$\begin{array}{lll} \text{I for web} & = \tfrac{3}{4}^{*}\cdot 1/12\cdot\tfrac{3}{8}\cdot 48\cdot 48\cdot 48 = 2590 \\ \text{I for 4 flge.Ls} & = 4\cdot 4.62(24.25-1.66)^2 = 9430 \\ \text{2 cov. pls.} & = 12\cdot\tfrac{7}{8}(24.25+0.22)^2 = 6310 \\ & \text{Total in inches,} \quad 18{,}330 \end{array}$$

$$S = Mc/I = 8{,}100{,}000\cdot 24.69/18{,}330 = 10{,}900 \text{ lbs. per sq.in.}$$

Spacing of horizontal rivets in flanges:

Statical moment: For 2 Ls $2\cdot 4.62(24.25-1.66) = 209$
1 Pl. $\tfrac{7}{16}\cdot 12(24.25+0.22) = 128$
Total, $\Sigma az = 337$

Distance from end, feet	0	6	12
Vertical shear per lin.in.	500	500	500
Shear	90,000	57,000	30,000
$\Sigma az/I = .0184$. Hor. shear per lin.in. $= .0184V =$	1,660	1,050	550
Resultant per lin.in.	1,730	1,160	740
Spacing = 4710/resultant	2.72	4.05	6.40

Spacing of vertical rivets:
Here there will be no vertical force.

Distance from end, feet	0	6	12
Shear as before	90,000	57,000	30,000
$\Sigma az/I = \dfrac{128}{18330} = .00698$			
Hor. shear per lin.in.	630	400	210
Value of two rivets in single shear 8640			
8640/shear per lin.in. = spacing	13.7	21.6	41.0

However, spacing must not exceed 6″ or $4\tfrac{1}{2}$″ if staggered (Art. 50 *e*1).

In above problems it has been assumed in all cases that top flange is properly supported. (Art. 50 *d*11.)

Tables in handbooks are often a great help in the computa-

* Deducting ¼ for holes.

tion of beams. Capacity under uniform loading for all the shapes and some girders may be found in Cambria or Carnegie.

Many other examples will be found in Vol. II. Especially important are those given in the Chapters on Plate Girders and Office Buildings.

Art. 55. Design of Tension Members

These may be of round rods, square rods, rectangular bars, angles, and built-up shapes. There are three ways in which the stresses due to its own weight may be combined with tension.

(*1*) Add maximum flexural stress to the tensile unit stress.

(*2*) Take account of the fact that the weight and pull cause a deflection at the center, at which point the moments are of opposite kind. (Fig. 55*a*.)

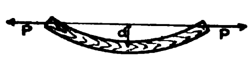


FIG. 55*a*.—Tie Acted upon by its Own Weight.

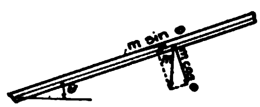


FIG. 55*b*.—Weight in an Inclined Member.

(*3*) Consider the varying eccentricity of the pull.

Many engineers ignore these stresses largely or wholly. (*1*) is good enough for comparatively short and deep sections. (*3*) is too complicated to be used in practice and is a needless refinement.

Taking up the second method, let w be the weight of the bar per lineal inch, I its moment of inertia in inches, E its modulus of elasticity in pounds per square inch, a its area in square inches, and l its length in inches. The maximum moment occurs at center of span and is:

$$M=(wl^2/8)-Pd.$$

Deflection due to a bending moment

$$(m=wl^2/8)$$

is

$$5wl^4/384EI=(m)(5l^2/48EI).$$

Assuming now that the deflection, d, bears a similar ratio to its moment, M:

$$M=(wl^2/8)-5Pl^2M/48EI \quad \text{or} \quad M=wl^2/8(1+5Pl^2/48EI)$$

$$S=Mc/I=wl^2c/8(I+5Pl^2/48E).$$

For a rectangle this becomes,

$$S=3wl^2h/(4bh^3+5Pl^2/E).$$

where b = breadth and h = depth, both in inches.

To obtain maximum stress, this must be added to direct stress.

In the case of an inclined member, Fig. 55b, weighing w per lineal unit, let any elementary weight, m, be resolved into components $m \sin \theta$ causing direct stresses which are small and may be ignored, and $m \cos \theta$ causing bending stresses. The maximum bending moment will occur at the middle and be equal to

$$\frac{wl^2 \cos \theta}{8},$$

where l is inclined length of member.

Square or round rods are easily fabricated and erected. They are usually adjustable and hence may work loose. Also their lack of stiffness makes them likely to rattle. Square rods can be turned with an ordinary wrench whereas a pipe wrench is required for a round. However, the latter fact may be an advantage if they are within ready reach.

Let it be required to design a steel rod for a tension of 46,000 lbs. Allowable stress, 12,000 lbs. per sq.in. E = 30,000,000 lbs. per sq.in.

Area required = 46,000/12,000 = 3.83 sq.in. Use 2″ square or $2\frac{1}{4}$″ round. Supposing now the square rod to be 20 feet long, let us investigate stress due to own weight which is 1.13 lbs. per lin.in.

$$S=\frac{3wl^2h}{4bh^3+5Pl^2/E}=\frac{3\cdot 1.13\cdot 240\cdot 240\cdot 2}{4\cdot 2\cdot 8+(5\cdot 46{,}000\cdot 240\cdot 240)/30{,}000{,}000},$$

= 774 lbs. per sq.in. and the rod should be made $2\frac{1}{16}$″ square instead.

For locations where members will always be in tension, rectangular bars are now the accepted design. Where idle or likely to take some compression, the best practice favors using compression shapes. Sometimes bars themselves are " counter-braced " that is, fastened together to resist some compression. These bars have their ends enlarged to provide connection for pins and are then known as eyebars, Art. 43. Thickness should not be more than $\frac{1}{4}$ its depth in order to give a well packed joint; it should not be under 1/7 to prevent weakness on compression side as a beam under its own weight.

Let us design an eyebar for a pull of 88,000 lbs. and an allowable stress of 15,000 lbs. per sq.in. Bar is 30 feet long, and inclined at an angle of 30 deg. with horizontal. Use first method.

88,000/15,000 = 5.87 sq.in. area required.

Try a $6'' \times 1\frac{1}{16}''$, area = 6.37 sq.in. Wt. per lin.in. = 1.81 lbs.

Direct stress = 88,000/6.37 = 13,800 lbs. per sq.in.

Bending moment $= wl^2 \cos \theta/8 = 1.81 \cdot 360 \cdot 360 \cdot .866/8.$

= 25,400 in. lbs.

$S = 6M/bd^2 = 6 \cdot 25,400/1.06 \cdot 6 \cdot 6 = 4000$ lbs. per sq.in.

As this stress will remain constant for the same depth, we can increase width and allow 11,000 lbs. per square inch for direct stress.

88,000/11,000 = 8.00 sq.in., use $6'' \times 1\frac{3}{8}''$.

Probably a 7″ bar would be more economical.

Angles, singly or in pairs, are used a great deal, either for wind bracing or small trusses. They do not bend or rattle like rods and they will stand some compression. Let us design a pair of angles for the same data as the square rod.

46,000/12,000 = 3.83 sq.in. Use 2 Ls $5'' \times 3'' \times \frac{5}{16}''$.
Gross area, 4.80 sq.in.
Deducting 2 holes, each $\frac{7}{8}'' \times \frac{5}{16}''$ for $\frac{3}{4}''$ rivets:
Net area = 4.26 sq. in.
Direct stress = 46,000/4.26 = 10,800 lbs. per sq.in.

Stress due to its own weight, assuming longer legs vertical;

$$1.37\cdot 240\cdot 240\cdot 1.68/8\left(12.52+\frac{5\cdot 46{,}000\cdot 240\cdot 240}{48\cdot 30{,}000{,}000}\right)=760 \text{ lbs. per sq.in.}$$

Total stress is $10{,}800+760=11{,}560$ lbs. per sq.in.

There are two common types of the built-up tension member, the I, Fig. 56*f* or *g*, and two channels, either rolled, Fig. 56*j* or *k*, or built up as in Fig. 56*l* or *m*. The former consists of four angles united either by a single continuous plate, by batten plates, or by lacing. Cover plates may be added top and bottom. The I is a favorite section for bracing capable of carrying compression, and for the web tension members of riveted trusses. The solid I should not be used where its web will be horizontal because so placed it retains water.

For heavier stresses, use one of the two channel sections. These are united top and bottom by lacing or batten plates. Flanges may be turned either way as determined by conditions at joints or clearance for riveting. Plates shown dotted may be added to increase strength.

As an example, let it be required to design in soft steel according to specifications, Art. 50, a built I section for a maximum tension of 142,000 lbs. and a minimum of 20,000. Member will be 25 feet long and vertical.

Allowable stress is $9000(1+20{,}000/2\cdot 142{,}000)=9650$ lbs. per sq.in.

Net area required $=142{,}000/9650=14.7$ sq.in.

Use 1 Pl. $14''\times\frac{7}{16}''$. Gross area 6.12 deduct for 2 $\frac{7}{8}''$ rivets.

4 Ls $5''\times 3''\times\frac{3}{8}''$. 11.44 " 4 "

Net area, 15.19 O.K.

Where a riveted section is horizontal or inclined, stress due to own weight may be very nicely taken care of by giving connection such an eccentricity that its moment balances that of the weight.

Art. 56. Design of Compression Members

The desirable features in a column are:

(*1*) Favorable disposition of the metal, that is, that disposition, which, for a given area, makes radius of gyration a maximum.

(*2*) Economy of shopwork. Columns are usually expensive to fabricate.

(*3*) Easy end and intermediate connections.

(*4*) Connections which give centrally applied loads. Even if eccentricities balance, live load on one side only may change this.

(*5*) In many places, it is desirable that the column should be as compact as possible.

(*6*) A section enclosed on all sides is objectionable, since it is inaccessible either for inspection or repair.

(*7*) Out of doors, all steel work should be so designed that it will not retain water.

Let us now consider the different types of columns in the light of the above. Taking up first, columns of a single shape.

(*a*) Angle. (*b*) Zee bar.

These have small radii of gyration and are limited to short columns and low stresses. Except for (*1*), they are desirable in every way.

(*c*) Channel.

This has a small radius of gyration one way, but a large moment of inertia the other. It is hence suited for small compressive stresses when combined with bending. Employed for struts at eaves of buildings and ends of bridges. Desirable in every way except (*1*).

(*d*) I-beam.

This is much like the channel but it has a larger radius of gyration and better withstands bending. We find it used for larger compressions and bending moments as in a column supporting the roof of a mill building. It is not as convenient for shopwork and connections on two sides are quite eccentric.

(*e*) H section.

The rapidity with which this column has sprung into favor

for moderate loads is explained by its advantages. The metal is well disposed, there is no shopwork except at connections and these are quite easy and fairly central, they are compact, are not enclosed and will not hold water unless placed horizontally. We know of but one objection to them, they cost about $0.20 per hundred weight more for the raw material.

Taking up now the built-up sections,

(*f*) Two angles riveted, Fig. 56*a*.

The piece between the angles shows a washer filler, Fig. 37*k*, which may be inserted at intervals or omitted altogether. This section is more economical if placed with the short legs outstanding. Its characteristics are much like those of a single angle column but the radius of gyration is a little larger and shop work is more expensive. Suitable only for short members and light loads. Employed for wind bracing and roof trusses.

(*g*) Two angles and a plate, Fig. 56*b*.

FIG. 56*a*
Two Angles.

FIG. 56*b*.
Two Angles and a Plate.

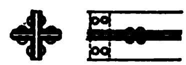


FIG. 56*c*.
Four Angles Riveted.

Column Sections.

Much like (*f*) except that it is designed to carry bending moment in addition to compression. Used for the top chord of roof and riveted trusses. This section might also be employed for tensile stresses.

(*h*) Four angles, riveted, Fig. 56*c*.

These are fastened together as shown with batten plates at intervals. It is economical of space, but metal is very poorly placed, shopwork is fairly high, connections are difficult and eccentric, and column is full of pockets. An undesirable section.

(*i*) Four angles latticed or battened. Figs. 56*d* and *e*.

The material of the section is well placed, but the proportion of details which do not carry weight is very high, shopwork is expensive, connections are difficult and costly. Used principally in long unbraced columns where it is important to keep weight low as in derricks.

(*j*) Built I-beam, latticed, Fig. 56*f*.

The shorter leg of the angle should be parallel to lacing for economy. Material is not well disposed and cost of details is high. It is used in locations where the stress is small and depth is demanded for sake of connections or to resist stresses due to own weight as in the bracing for a bridge.

(*k*) Built I-beam, solid, Fig. 56*g*.

Here there is less waste material than in (*j*). Plates shown dotted increase allowable unit stress as well as area. The radius of gyration, however, still continues small, and it is not a very economical section. Shopwork is moderate for either. Connections are easy but eccentric on two sides. Used as columns in buildings and in web members of riveted truss bridges.

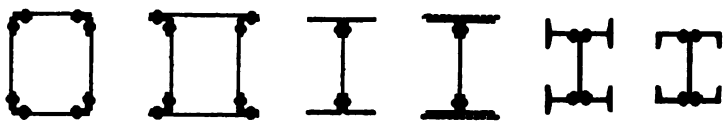


Fig. 56*d*. Fig. 56*e*. Four Angles Latticed or Battened.

Fig. 56*f*. Fig. 56*g*. Built I's Latticed Solid.

Fig. 56*h*. Three I Beams.

Fig. 56*i*. Two Channels and One I.

Column Sections.

(*l*) Three I-beams, Fig. 56*h*.

This section seems quite desirable but, on account of the slightly greater advantages of (*m*), is not common.

(*m*) Two channels and one I-beam, Fig. 56*i*.

As in (*l*), section is very well placed, it is economical of shopwork, and easy to connect with. While either is likely to have eccentric connections on two sides, they are well able to bear this. It is not a closed section, and it does not hold water if placed vertically. It does, however, occupy considerable room, (*l*) slightly more than (*m*), and former is also a little harder to handle. They are ideal where a moment in both directions is to be carried, as in columns for cranes and elevated railroads. Obviously, either (*l*) or (*m*) may be built up.

(*n*) Two channels, latticed, Figs. 56*j* and *k*.

Metal is well placed but shopwork and weight of details are moderately high. Connections are fairly easy and this type is frequently used as, for example, in parts of bridges

and columns for buildings. Extra plates may be added either outside or inside as shown by dotted lines.

(*o*) Two built channels, latticed, Figs. 56*l* and *m*.

Very much like (*n*) except that shopwork is higher and that it is used where the rolled shapes will not provide the necessary area.

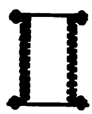
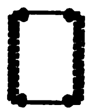
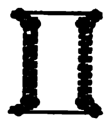
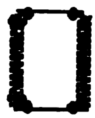
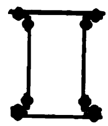
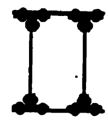







FIG. 56*j*. FIG. 56*k* FIG. 56*l*. FIG. 56*m*. FIG. 56*n*. FIG. 56*o*.

Two Channels, Latticed. Two Built Channels, Latticed. Two Channels and Cover Plate.

Column Sections.

(*p*) Two channels and a cover plate, Fig. 56*n*.

Channels are turned as shown and may be rolled or built-up. Plates may be added as in (*o*) or, if nothing interferes, angles may be added on the inside, Fig. 56*o*. Material is well placed, shopwork moderate, open for inspection, and especially suited for easy and efficient connections in the top chords of bridges where it is the accepted design.

(*q*) Built I and two built channels, Fig. 56*p*.

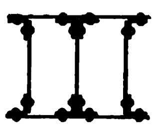


FIG. 56*p*
Built I and Two Built Channels.

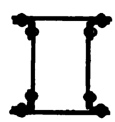


FIG. 56*q*.
"Box."

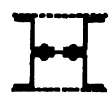


FIG. 56*r*.
Zee Bar.

Column Sections.

To still further increase capacity, one or more built I's may be inserted between channels in (*o*), and in a similar way extra plates could be added. As in (*p*), one might rivet on a cover plate in place of lacing.

(*r*) Box column, two channels and two cover plates, Fig. 56*q*. Channels are either rolled or built up. Material is very well placed, shopwork moderate, connections easy, but they are quite eccentric, and it is a closed column. Used in building

work, where it should always be incased in fireproofing to ensure against rust.

(*s*) Zee-bar column, Fig. 56*r*.

Here, the theoretic disposition of the metal is very good, but under test the outside corners fail first because they are insufficiently stayed. Connections are central and easily arranged. Is accessible for inspection except when plates shown dotted are added to secure increased strength.

Among the remaining types of columns, we may mention the Phœnix, now obsolete, and the Larimer, Fig. 22g, both of which are explained in Art. 22.

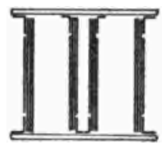


FIG. 56*s*.—Section of the Bottom Chord of the Quebec Cantilever Bridge.

As an example of how large columns are built up, we give Fig. 56*s*.* This shows a section of the bottom chord of the Quebec cantilever bridge, the failure of which caused the wreck of the structure. (Art. 69.) The outer ribs were made of 2 L's, $8'' \times 6'' \times \frac{15}{16}''$, 1 Pl. $54'' \times \frac{7}{8}''$, 2 Pls. $54'' \times \frac{13}{16}''$, and 1 Pl. $37\frac{3}{4}'' \times \frac{15}{16}''$. The inner ribs were 2 L's, $8'' \times 3\frac{1}{2}'' \times \frac{15}{16}''$, 2 Pls. $54'' \times \frac{13}{16}''$, 2 Pls. $46'' \times \frac{15}{16}''$. Lacing was double, 45 deg., of 1 L $4'' \times 3'' \times \frac{3}{8}''$ and cross struts, 1 L $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$.

In design, the author prefers the straight line formula,

$$P = C_1 A(1 - C_2 l/\rho).$$

where P is total load and A gross area; l/ρ, the greatest slenderness ratio, that is, the greatest value of the fraction, unsupported length divided by corresponding radius of gyration. C_1 is the unit strength for a short strut and C_2 is a constant for a given material which will bring the straight line tangent to Euler's curve for long columns. We have made C_2 a little large, about .006, in order to discourage the use of long slender columns.

The stress due to own weight may be taken care of as in tension members. However, the moment due to the load P (see preceding article), is plus instead of minus and hence the formula becomes:

$$S = wl^2c/8(I - 5Pl^2/48E).$$

* Engineering News, Vol. LVIII, p. 320.

It is customary in main members to give the connection just enough eccentricity to balance the stress due to own weight as already mentioned for ties.

Let us take up now the fastening together of the parts of a built-up column. In the first place, the distance between the points at which the parts are riveted should be such that the slenderness ratio for none of those parts exceeds that for the column as a whole.

To obtain strength required for this fastening whether of lacing or rivets, let us consider the above formula,

$$P = C_1 A(1 - C_2 l/\rho).$$

The reason why long columns fail at lower unit stresses than short ones of the same cross-section is that irregularities of manufacture cause an eccentricity, the effect of which is similar to a uniform load applied at right angles to the column. The transverse load causes a bending with compression on one side and tension on the other, and post fails when sum of compressions due to flexure, S_1, and axial load, S_2, reaches C_1.

$$C_1 = S_1 + S_2 = S_1 + C_1(1 - C_2 l/\rho)$$

or

$$S_1 = C_1 C_2 l/\rho.$$

But

$$S_1 = Mc/I = Wlc/8A\rho^2,$$

assuming that load which causes bending moment is uniform. Where W = uniform load applied transversely, and

c = distance from neutral axis to most strained fiber.

Equating these two values of S_1, and substituting $4\rho/3$, a rough value, for c,

$$W = 6AC_1C_2.$$

Taking for C_2, the value .006,

$$W = .036\ AC_1,$$

or about one-thirtieth load on a short strut of same section. Some of the assumptions made in the proof might be questioned on theoretic grounds but the fact remains that the rule here deduced agrees very well with present practice.

In addition to specifications in Art. 50, *e*3, and *e*4, the following should be noted. Lacing may be single, inclined at an angle of about 60° with axis, and, if there are two sets, staggered as shown in Fig. 56*t*; or double, at an angle of 45°, Fig. 56*u*. In either case, it should be figured as a truss. However, as it is more economical to make all pieces alike, only

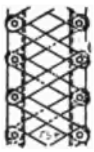


FIG. 56*t*.

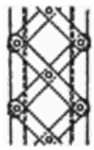


FIG. 56*u*.

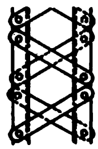


FIG. 56*v*.

Arrangement of Lattice Bars.

FIG. 56*w*.

Showing Use of Batten Plates.

maximum stress in end member need be computed. Lattice bars are often placed as in Fig. 56*v*, but it does not seem like an effective arrangement; we must admit that it is employed in good work. As already mentioned, occasional batten plates are sometimes inserted in place of lacing, Fig. 56*w*; nevertheless, as may be inferred from above analysis, it is inefficient and unsuitable for important locations.

The lattice bars vary in size from $1\frac{1}{2}'' \times \frac{1}{4}''$ to $2\frac{1}{2}'' \times \frac{3}{8}''$ and even larger for built-up columns. Angles may be used to

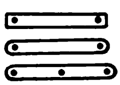


FIG. 56*x*.

Forms of Lattice Bars.

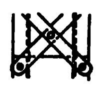


FIG. 56*y*.

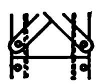


FIG. 56*z*.

Arrangements of Batten Plates and Lattice Bars.

advantage in very large compression members. Fig. 56*x* shows different ways of making the bars. The center of the curve is seldom at the rivet but nearer the outer end.

The latticing should start either from the batten plates, Fig. 56*y*, or from a rivet as close to it as clearance will allow, Fig. 56*z*. Lattice bars may be used where the surfaces with which they connect are not in exactly the same plane. For example, the lattice bar in Fig. 56*y* might run from on top of the batten plate to top of channel.

Let it be required to design a medium steel single shape column 8 ft. long according to specifications in Art. 50. Load is 50,000 lbs. dead.

Allowable stress, 15,000 $(1-.006\ l/\rho)$.

Maximum $l/\rho=100$, minimum $\rho=0.96''$.

All zee bars and channels have lower radii of gyration, but we can use any equal legged angle above a $4''\times4''$ and any I beam above $10''$ in depth.

Section.	Weight per Foot.	ρ	Allow. Stress.	Area.	Total Stress.	Remarks.
*1L 5×5× 7/8	27.2	0.96	6000	7.99	48,000 #	Too low
*1L 5×5×15/16	28.9	0.96	6000	8.50	51,000 #	O.K.
1L 6×6×11/16	26.5	1.17	7620	7.78	59,300 #	Too high
1L 6×6× 5/8	24.2	1.18	7680	7.11	54,600 #	O.K.
1L 6×6× 9/16	21.9	1.18	7680	6.43	49.300 #	Too low
1L 8×8× 1/2	26.4	1.58	9570	7.75	74,000 #	
1I 10.........	25.0	web too thin				
1I 12.........	35.0	0.99	6270	10.29	64,700 #	

Everything in pounds and inches.

Number 4 should be used. We need not have tried numbers 6 and 8, after we had found their weight unless a surplus of strength were desirable. In this case we should use 6, as it has a larger capacity and less material than 8.

Next we will compute the size required for a medium steel column of two channels latticed with flanges turned out. Specifications as in Art. 50. Post is vertical and 25 feet long. Loads range between 112,000 compression and 25,000 tension. Estimating allowable stress at 6000, about 19 square inches will be needed. Let us try 2 channels 15″at 33 lbs.

Allowable stress

$$=10,000(1-25,000/2\cdot112,000)(1-.006\cdot300/5.62)$$

$$=6040 \text{ lbs. per sq.in.}$$

Area required $=112,000/6040=18.5$ sq.in.

* Special sections and undesirable on that account. Also so thick it would have to be drilled (Art. 44).

19.8 are furnished, so this is O.K. There is plenty of metal for tension.

To design the latticing, let us take it as single and inclined at 60° with the axis. Gage to gage will be about 13.25″ and the unsupported length of the bars will be 13.25 sec 30° = 15.3″. Assuming $\frac{3}{8}$″ thickness, the allowable stress is,

$$10{,}000(1-25{,}000/2\cdot 112{,}000)\ (1-.006\cdot 15.3/.107)=1260 \text{ lbs. per sq.in.}$$

Transverse load is

$$(10{,}000/30)\quad (1-25{,}000/2\cdot 112{,}000)\quad (19.8)=5850 \text{ lbs.}$$

Stress at either end is, 5850 sec. 30°/4 = 1690 lbs.

1690/1260 = 1.34. Use $3\times\frac{7}{16}$″, area 1.31 sq.in. Amply safe with depth larger than assumed.

Now let us determine sizes for a two-channel and a plate section of medium steel, Fig. 56*n*. Length, 20 feet; maximum load 280,000 lbs. C; minimum load, 80,000 lbs. C. Specifications as in Art. 50.

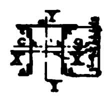


FIG. 56*aa*.

Assume allowable stress to be 9000 lbs. per sq.in.

Then area required will be 280,000/9000 equals 31 sq.in. Let us try section shown in Fig. 56*aa*, horizontal distance in clear between plates, 10″.

Size.	Area.	Distance to Center of Gravity.	Static Moment. XX	I Moment. XX	I Moment. YY
a. 1 Pl. $18\times\frac{3}{8}$	6.75	8.31	+56.1	466	182
b. 2 Pls. $16\times\frac{3}{8}$	12.00	00	00	256	323
c. 2 Ls. $3\frac{1}{2}\times3\frac{1}{2}\times\frac{3}{8}$	4.96	7.11	+35.3	257	212
d. 2 Ls. $3\frac{1}{2}\times3\frac{1}{2}\times\frac{5}{8}$	7.96	7.02	−56.0	402	335
For entire section....	31.67	1.12	+35.4	1381	1052

Everything in inches.

$$I_{XX} \text{ about c.g.} = 1381-31.67\cdot 1.12\cdot 1.12=1341.$$

$$\rho=(I/A)^{\frac{1}{2}}=(1052/31.67)^{\frac{1}{2}}=5.75 \text{ in.}$$

Allowable unit stresses

$$=10{,}000(1+80{,}000/2\cdot 280{,}000)(1-.006\cdot 240/5.75).$$
$$=8570 \text{ lbs. per sq.in.}$$

Total allowable stress

$$=31.67\cdot 8570=272{,}000 \text{ lbs.}$$

This is too low. We will use bottom angles $\frac{11}{16}''$ thick, making total quantities:

Area, 32.39; distance to center of gravity, 0.96″; statical moment about XX, 30.9; I about center of gravity, horizontal

FIG. 56*ab*.—Typical Column. Upper Chord of Pin-connected Truss Bridge, American Bridge Co., Ambridge, Pa.

axis, 1383; I_{YY}, 1096; ρ, 5.80″; allowable unit stress 8590; total allowable stress, 278,000 lbs.; will do.

Allowing 40% for extras, weight per foot will be 154 lbs. Deflection $=5Wl^3/384EI$

$$=5\cdot 3080\cdot 240\cdot 240\cdot 240/384\ 30{,}000{,}000\cdot 1383=.013''$$

M due to own weight $=3080\cdot 240/8=92400$ in. lbs.

Amount which the center of gravity of the column should be placed above the intersection point is,

$$.013+92{,}400/280{,}000 = .342 \text{ or about } \tfrac{3}{8}''.$$

It will be noticed that any additional plates that may be necessary will be located on the side at a distance from the center about equal to the radius of gyration. Therefore allowable unit stress will be changed little when reinforcement is tacked on. In practice, for a case like this, allowable stress need not be recomputed.

Latticing must be figured for a transverse load of,

$$10{,}000(1+80{,}000/2\cdot 280{,}000)\ 32.39/2\cdot 30 = 6200 \text{ lbs.},$$

the other half being carried by the top plate. Maximum shear is 3100 lbs. at each end. Latticing is usually made double, 45°, and riveted at the center. Stress in end bar is then 3100 sec. 45°/2 = 2190 lbs. Unsupported length is about $7\frac{1}{2}$ sec. 45° = 11″.

Let us try a $3\times\frac{5}{16}''$, $\rho=0.09''$. Allowable total stress equals 2860 lbs. O.K. A $2\frac{1}{2}\times\frac{3}{8}''$ would be as cheap and much more efficient.

While lacing bars are usually weakest in compression, rivets and tensile stresses should also be watched.

FIG. 56ac.—Typical Column. Vertical Post in Drawbridge, American Bridge Co., Ambridge, Pa.

Art. 57. Strain Sheet

"Stress Sheet" would be more appropriate but we follow custom.

The strain sheet gives a line drawing of proposed structure with its principal dimensions. There should also be a statement as to loads assumed, total stresses resulting therefrom, unit stresses allowed, and sections designed for the different pieces. Second and last are usually placed directly on the members to which they belong. Title should give name and location of designer and purchaser. Date, scale, and name of maker ought also to appear on the drawing. For lettering, see Art. 62, also sample strain sheet, Fig. 57.

Sometimes important or peculiar connections are drawn out. Often all details are worked up, and leading dimensions and material given. It partakes then of the nature of a detailed drawing and is called a "general plan."

Art. 58. Detailing

Detailing may be divided into two parts:

(*a*) The design of the small parts of the structure—rivet spacing, connections, shoes, and so on, which will be taken up in Arts. 59, 60, and 61, and also in other volumes.

(*b*) Structural steel consists of rolled shapes, cut, forged or bent, punched or drilled, and machined. Specifying and locating these form the second part. See also Art. 40.

There are two general methods of accomplishing (*b*). Take, for example, a roof truss. In the first method, we show by sketch number of rivets and method of arranging connections, give center to center distances, state material required, also maximum and minimum spacing and edge distances and leave templet shop to arrange details on full size layout on shop floor. This method is cheap, especially in the drawing room, but it renders control by the engineer more difficult. Also in case of repairs or alterations, expensive measurements on site may be necessary.

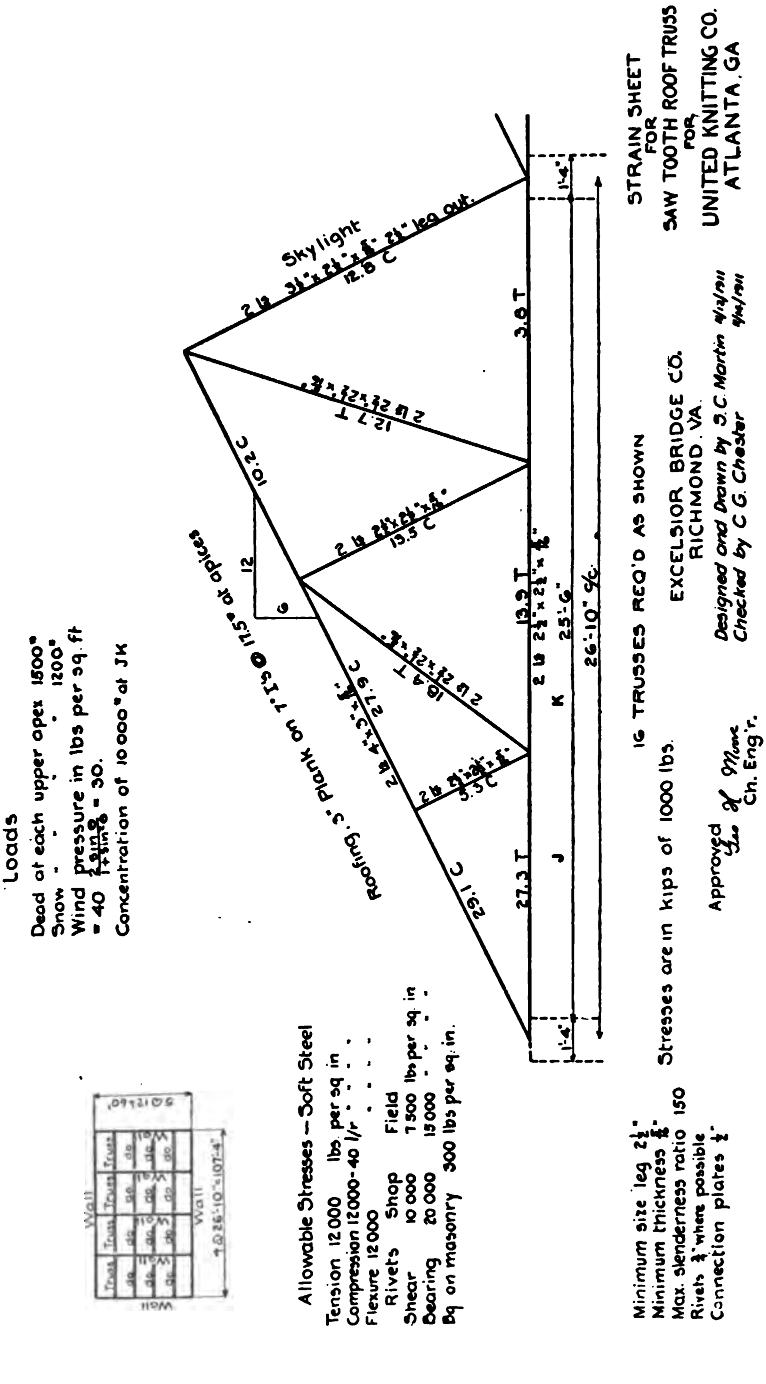
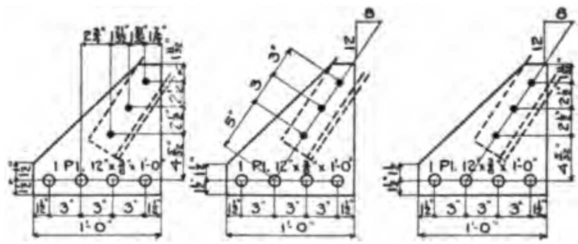

Loads
Dead at each upper apex 1500#
Snow " " " " 1200#
Wind pressure in lbs per sq. ft
= 30.
Concentration of 10000# at JK
Allowable Stresses — Soft Steel
Tension 12000 lbs. per sq. in
Compression 12000-40 l/r " " " "
Flexure 12000 " " " "
Rivets Shop Field
Shear 10000 7500 lbs per sq. in
Bearing 20000 15000 " " " "
Bg on masonry 300 lbs per sq. in.
Minimum size leg
Minimum thickness
Max. slenderness ratio 150
Rivets where possible
Connection plates
Skylight
12.8 C
3.8 T
12.7 T
10.2 C
13.5 C
13.9 T
18.4 T
27.9 C
5.5 C
27.3 T
29.1 C
12
6
J
K
25'-6"
26'-10" c/c
1'-4"
Stresses are in kips of 1000 lbs.
16 TRUSSES REQ'D AS SHOWN
EXCELSIOR BRIDGE CO.
RICHMOND, VA.
Designed and Drawn by S. C. Martin 4/12/1911
Checked by C. G. Chester 4/14/1911
Approved
Ch. Eng'r.
STRAIN SHEET
FOR
SAW TOOTH ROOF TRUSS
FOR
UNITED KNITTING CO.
ATLANTA, GA

FIG. 57.

The second method gives all dimensions required to work out each piece separately. Fig. 58*a* shows three different ways in which this may be done for skew measurements. (*1*) is more expensive and liable to error and is used very seldom and only to prepare for rack punch. (*2*) and (*3*) are both good;



(1) (2) (3)

$\frac{7}{8}''$ rivets $\frac{15}{16}''$, open holes.

FIG. 58*a*.—Methods of Detailing Connection Plates.

latter is a little easier in drawing room and a little more difficult in shop.

There is still another subdivision. Structures may be detailed "in place," that is, with the pieces in the relative position which they will occupy in the field. This is easier to understand, but the manual drawing is more difficult and it takes additional space. Or "knocked down" with pieces taken apart and given in their most convenient position. Fig. 58*b* shows difference. It is not common to take apart a shipping piece.

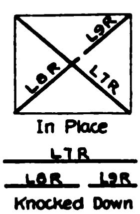


FIG. 58*b*.—Methods for Laterals.

Consider now the simple cases given above. Fig. 58*a* shows details of a plate; *c*, an angle; *d*, an I-beam; and *e*, a plate girder. To enable shops to fabricate these or other structures;

(*1*) Bill material. Standard fittings need only be named. For example, rivets are specified by diameter, given in note as shown. Other pieces of metal must be billed as mentioned in Chapter II. This material is lettered on the drawing in such a place that there will be no doubt as to which piece is

intended. Often a table is made at the side giving the information, but unless an assembly or identifying mark is used, or

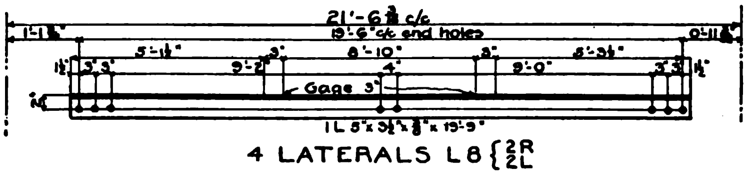


FIG. 58*c*.—Detail of an Angle.—Open Holes, $\frac{13}{16}''$.

except for very simple structures, the writer does not approve. It is best to place notation as near piece as is feasible, with arrows if necessary.

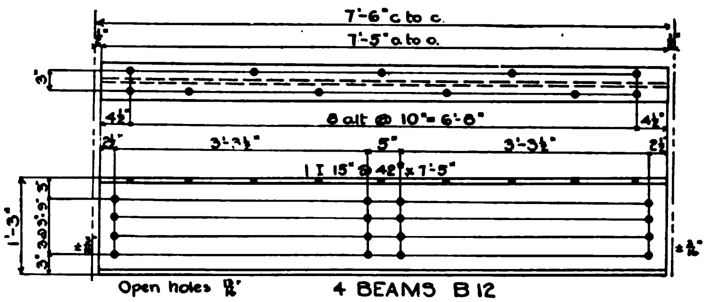


FIG. 58*d*.—Detail of an I-Beam.

(*2*) In these shapes, holes will be drilled or punched. Longitudinal spacing must be given in all cases. If more than

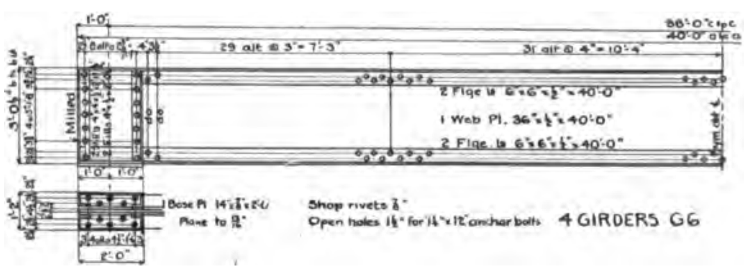


FIG. 58*e*.—Detail of a Plate Girder.

three are alike, they should not be repeated, 3″, 3″, 3,″ 3″, 3″, but marked 5 at 3″ = 1′–3″. Rivets alternately spaced on two

lines as seen in Fig. 58*e* and in top view of Fig. 58*d* are located so many alternate spaces. Note that distances given are not the center to center measurements but are taken along dimension line from one rivet to a point opposite the next.

Suppose now we have an angle 20 feet long with 40 rivets, 6″ apart. Assume first that the angle goes in a space just 20′–0$\frac{1}{8}$″ long. Over all distance or two edge distances must then be marked "not more." Second, suppose piece is to be machined to be 20 feet long when finished. Both ends are then marked "mill," both edge distances given, and shop takes care of it if enough material is ordered. Third, let variation either way of a quarter inch be unimportant. Practice varies here, two, one, or none of the edge distances being given, as it is left largely to the discretion of the shop whether a piece should be cut to exact length or not.

This brings us to the subject of mill variation, see Art. 17. Shapes from the mill do not come exact length unless a prohibitory price is paid. The schedule of variation is quite complicated. We will state but one very important item,—I-beams and channels may come $\frac{3}{8}$″ long or short. Hence we detail them so this uncertainty will do no harm and mark the allowable variations, usually $\frac{3}{16}$″, on each end, Fig. 58*d*. In case it is required to mill the end of an I-beam or channel, it must be ordered long enough so there will be sufficient metal even if it comes $\frac{3}{8}$″ short.

(*3*) Next transverse spacing must be given. For each size I-beam, angle, and channel, there are certain standard spacings (see hand-book). These are determined by edge distance and clearance for driving. In case there is more room than needed, variation may be made from the standard. However, this should be done only by an experienced draftsman.

Fig. 58*f* shows how dimensions are given for shapes other than rectangles. In I-and T-beams, spacing in flanges is symmetrical. If dimensions marked x are given, it means shape is to be milled or cut to this dimension. Avoid, if possible, as it is very expensive.

(*4*) Cuts may be located by three methods as illustrated in Fig. 58*g*.

(*5*) Bends are specified by bevels, by radius of curvature

and bevels, or by the dimensions of the piece on which it goes.

(*6*) Overall and center to center dimensions of finished piece must be given. Distances between groups of field rivets are also important. Besides their convenience in the drawing room, they are an aid to the inspector. Always give back to back of angles. In case of long members with complicated spacing, locate intermediate points by a separate line of dimensions.

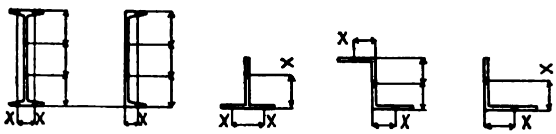


FIG. 58*f*.—Transverse Spacing of Shapes.

Let us take up now the question of assembly marks. Suppose a job involves 1000 tons and 60 sheets of drawings. Certain details exactly alike and forming part of pieces to be shipped occur on different plans, often several times on the same sheet. These may be handled in the ordinary way or method of assembly marks may be used. The latter system is about as follows:

Give all except main members a mark. Those on first sheet will be *a1*, *b1*, . . ., *aa1*, *ab1*, . . ., *ba1*, *bb1*, and so on. If

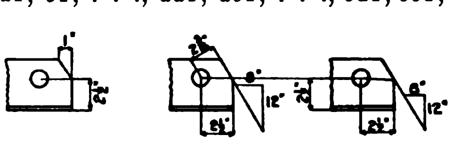


FIG. 58*g*.—Methods of Detailing Cut.

now on sheet two, a part occurs which is the same as on one, it is given its old mark, *am1* for instance. Details not like anything previous will be given new ones, *a2*, *b2*, . . ., *aa2*, etc. Parts that are alike have the same mark and conversely. Those which are right and left must be so designated. Details are given where piece first occurs; elsewhere only enough is furnished for other connecting parts; except that material is billed once on each sheet which contains it, thus, 1L, $4'' \times 3'' \times \frac{3}{8}'' \times 1'-0''$ *bm3*. Elsewhere on sheet, it is simply *bm3*. In bills of material, *bm3* is listed in every place where it occurs. At the first of these

places, total number of *bm3* is given. In the templet room, the workman who is assigned to sheet 3 makes templet; if it occurs on other sheets, the templet maker knows from its number that it has already been taken care of. It causes some additional work in the drawing room but is economical and efficient in the shop.

Every piece which is shipped must have its mark for identification during erection. This should be suggestive, *G* for girder, *S*, stringer, and so forth. In trusses, joints are sometimes lettered U_0, U_1, etc., above; L_0, L_1, etc., below. $U_1\ L_0$ is then the endpost. These marks should be given directly under member and in letters somewhat larger than rest of drawing. Writer prefers the form,

$$5 \text{ Girders } G14 \left\{ \begin{matrix} 3R \\ 2L \end{matrix} \right. .$$

Among the many other ways in which it may be written, we mention,

$$5 \text{ Girders required} \left\{ \begin{matrix} 3 \text{ as shown mark } G14R \\ 2 \text{ other hand mark } G14L. \end{matrix} \right.$$

But every experienced man knows what the first inscription means and no additional information is given to pay for extra space and time consumed by the second.

To distinguish between assembly and shipping marks, observe that several of the former are riveted together to make one shipping mark, and a number of latter when fastened together form finished structure.

Important points to be borne in mind in detailing are:

(*1*) Dimensions must be accurate to $\frac{1}{32}''$.

(*2*) All necessary measurements must be given, but,

(*3*) Avoid needless repetition.

(*4*) Everything must be clear and concise.

(*5*) Show connecting work detailed on other sheets in red. And in the design of details,

(*6*) Use as few shapes as possible in addition to those employed for sections.

Art. 59. Design of Splices and Beam Connections

The following principles are important for all joints, whether splice, riveted connection, or pin joint:

(1) The joint should be economical of material and shop-work and erection, the latter two factors being the more important.

(2) Be careful to make details of sufficient strength. Two methods of computation are employed:

(a) Make details as strong or stronger than the main members.

(b) Make details as strong as stresses. Sometimes for small stresses, specifications for minimum sized material make much larger members obligatory. The difference between (a) and (b) is then considerable. The author prefers to use (a) when the structure thus strengthened is worth enough more to pay for the additional outlay.

(3) Consider erection very carefully.

(4) Make joints as rigid as possible.

(5) Compactness adds to rigidity, economy, and strength.

(6) Important members should meet at a point except as necessary to balance moment due to own weight. (See Arts. 55 and 56.)

In addition for riveted joints.

(7) Keep field riveting to a minimum as already noted. (Art. 49).

(8) One rivet is not enough and two are too few for important work.

Practice has established a number of conventional rules for the computation of rivets which do not accord with actual conditions. Nevertheless, customary methods seem safe since constants are derived from tests making same assumptions. These are:

(a) That rivets completely fill the holes; as they are driven hot and afterwards shrink, this cannot be true.

(b) While considered to fill the holes, their capacity is computed from the original diameters.

(c) Although supposed to carry their stresses by shearing

and bearing, they actually hold by the friction of the cooling rivet.

(*d*) While we neglect bending, there must be considerable in the body of the rivets.

(*e*) In tension, stress is considered as uniformly distributed over net area with holes $\frac{1}{8}''$ greater than nominal diameter of rivets.

(*f*) Compression is considered as distributed uniformly over gross area if rivets are driven; otherwise over net area.

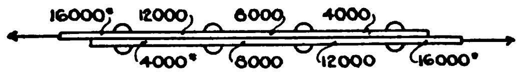


FIG. 59*a*.—Assumed Distribution of Stress.

(*g*) Shear is sometimes taken as distributed over net area and sometimes as over gross; the former seems more logical.

(*h*) That the stress in a group of rivets is uniformly distributed is far from the truth. First, the end rivets of a straight axial connection must carry more than middle. If stresses in rivets were equal, we would have impossible conditions, shown in Fig. 59*a*. Secondly, loads are often applied eccentrically. Let us suppose a load of 24,000 lbs. to be applied to the group of rivets shown in Fig. 59*b*. The stress assuming equal distribution is now 4000 lbs. in each rivet. Suppose, however, that load is applied $1\frac{7}{16}''$ to right of right hand row of rivets. The eccentric application of the load now causes a moment of $24{,}000 \cdot 2\frac{9}{16} = 62{,}000$ in.-lbs. Let R be the stress on the 4 outer rivets; then that on the other two will be $1.12R/3.2$, assuming stress to vary as distance from center of gravity of group. The moment of $4R$ is $4 \cdot R \cdot 3.2 = 12.8\ R$. That of the other two is $2 \cdot 1.12 \cdot 1.12 \cdot R/3.2 = 0.78R$. Total $= 13.6\,R = 62{,}000$.

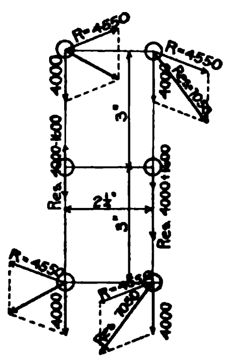


FIG. 59*b*.—True Stresses in a Connection Angle.

$$R = 4550 \text{ lbs. stress in outer rivets.}$$

$$1.12R/3.2 = 1600 \text{ lbs. stress in other rivets.}$$

Maximum will be the resultant of 4000 and 4550 = 7050 lbs.

Joints may be divided as follows:

(*a*) Splices
- (*1*) Tension.
- (*2*) Compression.
- (*3*) Bending.

(*b*) Beams connecting to other pieces. Subdivided according to position with regard to other piece.
- (*1*) Resting upon it.
- (*2*) Framing into it.
- (*3*) Suspended from it.

(*c*) Truss connections
- (*1*) Riveted.
- (*2*) Pin.

(*c*) Will be the subject of the next article.

In (*a1*) and (*a2*), it is customary to splice by providing plates on all sides, each with not less than two rows of rivets. For the former, Fig. 59*c*, number of rivets must be figured. In the latter, Fig. 59*d*, we may (*1*) shear ends and compute the

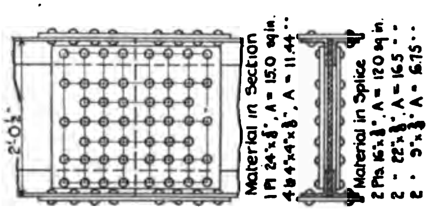


FIG. 59*c*.—Splicing a Tension Member.

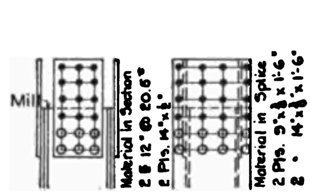


FIG. 59*d*.—Splicing a Column.

necessary rivets, (*2*) mill ends and put in about two or three rows of rivets each side of the joint as shown in the figure, or (*3*) design rivets to be as strong as member in bending. Common practice is to use (*2*) and make splice as near a support as connections will permit.

(*a3*) Splicing of beam occurs seldom except for plate girders, under which head it will be taken up in detail. To illustrate method, let it be required to splice a 24″ I at 80 lbs. to conserve its full net strength. See Fig. 59*e*. On each side, we will use a $20'' \times \frac{3}{8}''$ plate: I, gross = 250; I, net, estimated = 175 in.^{4} for each. I/c for a 24″ I at 80 lbs. = I/c for splice = 174.0. Estimating c at 13.0″, necessary $I = 2262$, or 1762 must be furnished by cover plates. Assuming their center of gravity to be at 12.5″, A for one flange is $1762/2 \cdot 12.5 \cdot 12.5 = 5.65$ sq.in.

Use two plates each $8'' \times \frac{3}{8}''$, gross area 6.00, net, 4.50 sq.in., deducting 2 1" holes. This computation equates the resisting moment of gross areas which is only approximately correct. The rivets in this flange will be in shear. To develop 4.5 sq.in. there will be necessary $4.50 \cdot 1.5$, or 6.75 sq.in. shearing area.

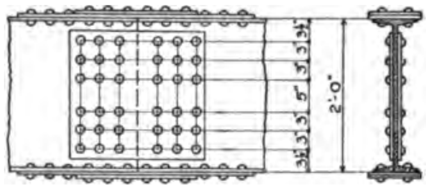


FIG. 59e.—Splicing a Beam.

This will mean 12 $\frac{7}{8}''$ shop rivets on each side of the joint as shown.

Theory for rivet splice at sides is similar to that for riveted connections just considered. Here the value R will be that for bearing and is equivalent to $\frac{7}{8} \times \frac{1}{2} \times 1.5 = 0.66$ sq.in. area for flexural stresses. 1.5 represents ratio of allowable bearing to bending if we assume shop rivets. In other words, one rivet is just as strong as 0.66 sq.in. at same point. $0.66(8.5^2+5.5^2+2.5^2) = 72 = I$ for half row of rivets. There will be required $350/144 = 3$ rows of rivets. The correct deduction for holes in side plates is $2(\frac{3}{8}) \cdot 1 \cdot (8.5^2+5.5^2+2.5^2) = 81$, leaving net I as 169, substantially as assumed.

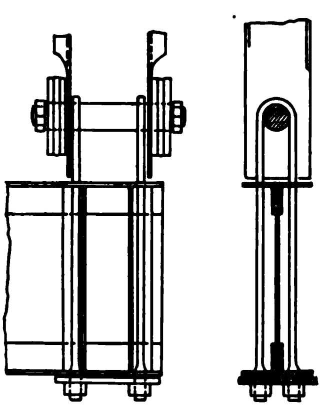


FIG. 59f.—Suspended Floor Beam.

Where a beam rests on top of a piece, ($b1$), it is preferably fastened thereto by holes through its base. The objections to this method are,—danger of buckling of webs which should be figured for compression and, if insufficient, stayed by stiffeners; lack of rigidity in beams which can be prevented by X bracing at each end; and want of support for piece to which it connects.

(*b3*) lacks rigidity and is now considered poor design. It is sometimes seen in the connection between the floor beams and trusses of old highway bridges, Fig. 59*f*.

The standard connection, (*b2*) is good. Fig. 59*g* shows common method. Sometimes a "shelf angle" is placed just below the beam and this, in conjunction with angles as in figure or an angle riveted onto top, forms an acceptable type. In latter case, upper angle must not be figured to carry any part of the load. Shelf angle alone makes an inferior connection for reasons given for (*b1*). When beam connects to a truss joint, it may fasten to one of the members, or to their prolongations, or be riveted direct to the plate. (*b2*) is the common style.

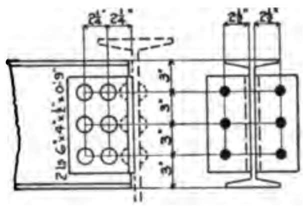


FIG. 59*g*.—Beam Connection.

Art. 60. Design of Riveted and Pin Joints in Trusses

The members of a truss which meet at a riveted joint are fastened together by means of one or two plates. The former is used for members composed of one or two angles. The latter when connecting to I beam and two channel sections. For this case, distance between plates at different joints should, if possible, be made the same and all truss members arranged to go outside or inside. For clearance, allow $\frac{1}{16}''$ or $\frac{1}{8}''$ if an entering joint, Fig. 49*m*, is necessary. Fastening plates to one of the pieces in the shop increases cost of shipping but lessens field rivets.

The thickness of plate unless determined by connections is preferably made such that resistance of rivets in shearing and bearing are about the same; however, it is seldom advisable to make it thick enough so material cannot be punched, Art. 44. Its length and breadth are determined by the necessary number of rivets. Plates are often irregular; if material cut from a rectangular plate will pay for extra labor (scrap is now, 1912, worth $\frac{1}{2}$ cent per pound), it is generally better to shear it off. (Art. 63.)

Let us next design joint shown in Fig. 60*a* which represents

a portion of a strain sheet. To get proper thickness of plates, t, we will equate bearing and shearing values of rivet, assuming unit stress for former to be twice the latter.

$$2 \times \text{Rivet area in bearing} = \text{Area in 2 shears, or,}$$

$$2td = 2\pi d^2/4 \text{ or } t = 0.79d.$$

$$d = 0.75, \text{ hence } t = 0.59'', \text{ use } \tfrac{9}{16}'' \text{ plate.}$$

Assuming allowable shearing and bearing values of 10,000 and 20,000 lbs. per sq.in. respectively, and for field rivets $\frac{3}{4}$ of

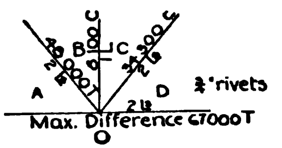


FIG. 60*a*.—Portion of Strain Sheet.

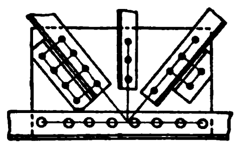


FIG. 60*b*.—Corresponding Detail.

these amounts, we may prepare the following table and then design our joint as shown in Fig. 60*b*.

Member.	Stress.	Rivet.	Value.	No. Rivets Required.
AB	48,000*T*	Field	6330	8
BC	8,100*C*	"	3310	3
CD	34,300*C*	"	6330	6
AD–DO	67,000*T*	Shop	8440	8

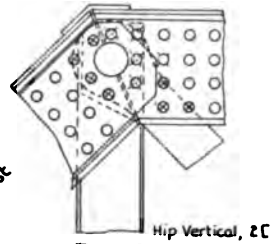
Everything in pounds. The last stress is really the maximum difference of stresses.

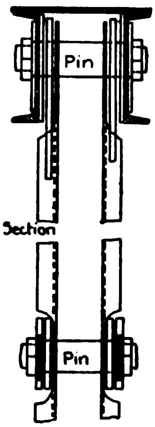
Pin Joints

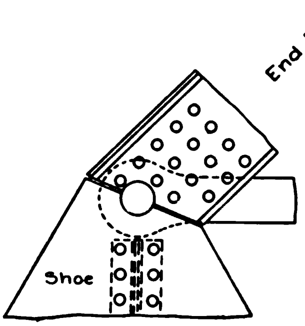
In the other type, the pin passes through the members directly. Compression pieces should be designed to facilitate this. Tension members are eyebars, and as these may be out of parallel by one-eighth of an inch in a foot, less difficulty is experienced in locating them. To explain how "packing plan" (a drawing showing arrangement of members around joints) influences design, let us examine typical joints of an ordinary Pratt truss. Figs. 60*c*, *d*, and *e* give packing plans for hip, shoe, and a panel point in the lower chord respectively.

The upper chords are made of such width that there is room for the diagonals outside the posts. The same position

FIG. 60c.—At Hip.







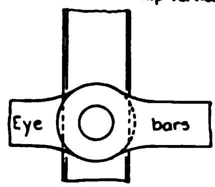


FIG. 60d.—At Shoe.

FIG. 60e.
At Lower Chord.

FIG. 60h.
Typical Riveted Joint.

FIG. 60i.
Typical Pin Joint.

American Bridge Co., Ambridge, Pa.

is maintained at bottom. The eyebars constituting bottom chord are passed outside the diagonals and inside end post. Webs of latter should be opposite those of shoe.

A pin is treated as a round beam acted upon by forces at various angles. We determine, often after several trials, which loading or loadings stress the pin most; then resolve all forces into horizontal and vertical components; find horizontal, vertical, and resultant shear, and horizontal, vertical, and resultant moment at salient points. From maximum resultant shears and moments, proper size of the pin may be determined by the usual rules of Applied Mechanics. The second volume of this work will give sample computations.

Pin connected structures are easier to erect. Also, their joints are more like the hinges assumed in computation, hence cause smaller bending stresses due to deflection. Riveted trusses deflect less and are stiffer.

Art. 61. Shoes*

Masonry is comparatively weak and is capable of bearing in compression but a very small part of load carried by an equal amount of steel. It is therefore necessary to build out at supports, and the structure for this purpose is called a shoe. It should be capable of transmitting the following forces:

(*1*) The vertical reaction from the girder to the masonry.

(*2*) A sidewise pressure caused by the wind or centrifugal force from the bracing to the anchor bolts.

(*3*) An uplift caused by wind from girder or truss to nuts of anchor bolts.

Short spans are sometimes fixed at both ends. The usual case, however, is that one end is fixed and the other is free to move. For the former it must carry:

(*4*) The longitudinal force caused by traction or application of the brakes.

We will next give desirable qualities for ideal shoes. First, for the free end:

(*a*) It should move with very little friction.

(*b*) It should be accessible for cleaning, oiling, or repair.

Next, for both ends:

(*c*) Arrangement ought to be such that bridge may be readily

* See Part III, "Details of Bridge Construction—Plate Girders," by Skinner.

detached from its foundation. As cinders, dirt, and moisture are likely to gather around the shoe, (*b*) and (*c*) need careful attention.

(*d*) Details should distribute pressure uniformly either before or after deflection.

The requirement (*d*) is usually ignored on short spans although the concentration of pressure caused thereby on the outside edge of the masonry must be considerable. For longer spans, we use the pin joint.

We shall classify as follows:

Type (*1*) Fixed without pin.
(*2*) Free without pin, sliding joint.
(*3*) Free without pin, rolling joint.
(*4*) Fixed with pin.
(*5*) Free with pin, rolling joint.

Type (*1*) Fixed without pin. (Fig. 61*a*.)

This is used for spans of less than 75 feet. Here a sole plate, $\frac{3}{4}$ to 1″ thick, is countersunk riveted to the bottom flange and placed upon a bearing plate of same thickness and dimensions. Planing is not necessary. Bolt holes about $1\frac{1}{2}''$ diameter for 2 anchor bolts $1\frac{1}{4}'' \times 12''$ or thereabouts are provided for fastening to the girder. In bridges built on a grade one of these plates is planed to allow therefor.

In a bridge where stringers are used, at each abutment, a shoe must be provided for every stringer or else a floorbeam be used at the end. The latter method is probably the better. It makes all the stringers alike and it cheapens the masonry, but it uses more steel, and the connections of the end floorbeam are often troublesome.

The bearing plate may be steel or cast iron. Where a high shoe is required, it may be made either of several plates riveted together or of cast iron, the latter detail being more frequent. In case the height is much in excess of 2 ins., it is usually made hollow. Underneath the sole plate is sometimes placed a sheet of lead and occasionally it is grouted up.

Type (*2*). Free without pin, sliding joint.

One method is to make as shown in Fig. 61*a*, the only difference being that both plates must be planed on their surface

of contact and the holes in the sole plate instead of being circular as in the fixed bearing are slotted as shown in Fig. 61*b*. The distance e is made equal to the expansion of span for maximum range of temperature. In this locality, (Pittsburgh) -20 to $120°$ F. are about the extremes. This gives us a variation of $(140 \times .0000065 = .00091) \times$ length. From this is derived an approximate rule; $\frac{1}{8}$ in. for every 10 feet. This allowance is also supposed to cover inaccuracies of fabrication and change in length due to deflection.

The distance d is made $\frac{1}{4}''$ larger than bolt for $1\frac{1}{4}''$ and smaller; $\frac{1}{2}''$ being added for $1\frac{1}{2}''$ or larger. Holes where $d = 1\frac{1}{2}$ and $e = 1''$ would be noted as,—"Slotted holes $1\frac{1}{2}'' \times 2\frac{1}{2}''$."

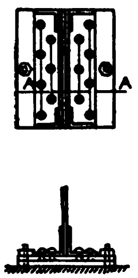


FIG. 61*a*.—Simple Fixed Bearing.

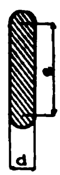


FIG. 61*b*. Slotted Hole.

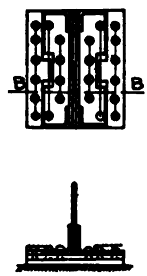


FIG. 61*c*.—Notched Expansion Bearing.

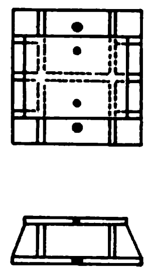


FIG. 61*d*. Cast Base.

Another method is to notch a plate into the flange or sole plate of the girder as shown in Fig. 61*c*. The notch in the flange should exceed in length parallel to bridge projection in the bearing plate by an amount equal to e, Fig. 61*b*. The plate shown dotted is advisable to provide against uplift. However, it is often omitted.

Either type (*1*) or (*2*) may be used in a simple form for the shoes of stringers, the slotted hole being the most common detail.

An example of a high cast-iron base is shown in Fig. 61*d*. This, it will be observed might serve equally well for a fixed end, the only difference being the holes in the sole plate which are slotted for the free end and round at fixed. Principles stated in Art. 16 must be carefully followed.

Bearing surfaces for cast iron should always be planed.

Sliding or rolling surfaces must be planed either for steel or cast iron. Sole plates are often omitted for short spans and cheap work.

Type (*3*). Free without pin, rolling joint. For spans over say 75 feet, friction caused by a sliding joint is too great and rollers (Art. 38) are substituted. For plate girders and small trusses these are usually 3 to 5″ diameter and circular. A large proportion of this cylinder does no useful work and, if cut away as shown in Fig. 61*e*, it continues to act as before, but occupies much less room to transmit a given pressure. At extremes of heat or cold, the rollers are inclined as shown by Fig. 61*e*; by cutting out as seen in Fig. 61*f*, we have another type which admits of larger expansion for a given distance center to center, or of closer spacing for a given expansion.

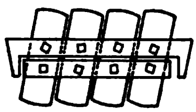


FIG. 61*e*.

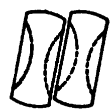


FIG. 61*f*.

Segmental Rollers.

There are two ways of providing for requirement (*2*), which calls for the transmission of forces acting horizontally and at right angles to the bridge. The first method is to fasten some shape, usually an angle, on the base plate so as to bear against the sole plate above as shown in Fig. 61*j*. If the fastening be tap bolts, the shapes may be readily removed so that the rollers can be cleaned, repaired, or oiled. It also helps to keep out cinders, dirt, and so forth.

In the second method, a strip about $2\frac{1}{2}'' \times \frac{1}{4}''$ is either riveted on or planed out from the bottom of the sole plate and the top of the bearing plate. A corresponding recess about $\frac{1}{16}''$ wider is turned in the rollers, Fig. 38*e*. It may be made exactly the same depth and then count as a part of the rollers in the computation for the length required; or, what is more common, a clearance of say $\frac{1}{16}''$ vertically may be allowed, but the width of the slot is then considered to carry no load.

Some means must be provided to keep the rollers the proper distance apart and parallel. For this purpose, circular rollers

need but one guide bar on each end. It may be either fastened to the rollers by tap bolts which are loose in the guide bars and tight in the female thread of the roller, Fig. 61*g*, or the roller may be turned down to a shoulder and enough allowed to project beyond the guide bar to allow a cotter pin to be inserted through a hole drilled for the purpose, Fig. 61*h*. Sometimes these shoulders are cut off flush with the edge of the guide bars

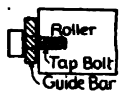


FIG. 61*g*.

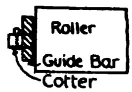


FIG. 61*h*.

FIG. 61*i*.

Details of Rollers.

and the latter are fastened together by two rods passing through pipes whose inside diameters are slightly larger than those of the rods between the bars, Fig. 61*i*.

In the case of segmental rollers, it becomes necessary not only to keep their axes, but also their planed faces parallel. The well-nigh universal way of doing this is shown in Fig. 61*e*. Clearance between upper and lower bars must be such as to allow the proper expansion. Rollers are tap bolted to each of the guide bars. The former are usually not less than 6″ high in order to allow room for two bars.

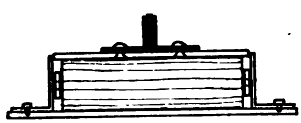


FIG. 61*j*.

FIG. 61*k*.

Roller Bearings.

Provision against uplift is frequently omitted altogether. It is best made, however, by prolonging the anchor bolts and passing them through longitudinally slotted holes in the sole plate, or by using Z bars, Fig. 61*k*, which while allowing the necessary longitudinal play hold it securely against any upward movement.

Type (*4*). Fixed with pin.

In the bearings thus far considered, the deflection of the girder, if originally true, concentrates a large part of the pressure

on the bridge end of the abutment. This may be prevented by giving the girder a camber, that is, an upward curve sufficient to make girder true when fully loaded. But the pressure is uniformly distributed then only for one loading.

A much better method is to use a pin bearing. The shoe is made in three parts. The upper part is firmly fastened to the girder and has one or more ribs arranged symmetrically about the center line of the girder, a diagrammatic view being given in Fig. 61*l*, which also shows the pin perpendicular to the plane of the girder carrying the stress which it receives to the lower part. This is generally much like the upper portion and rests directly on the under surface which as well as the top surface of the shoe should be planed. Transverse " diaframs," explained below, would be necessary for shoe seen in Fig. 61*l*. These

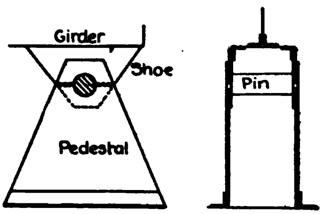


FIG. 61*l*.—Pin Bearing.

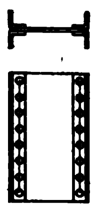


FIG. 61*m*.—Diafram.

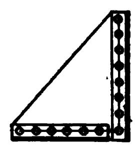


FIG. 61*n*.—Gusset.

are not shown in drawing. They must be computed to carry entire load as shoe would otherwise have little capacity. In pedestal, they are needed to stiffen plate.

The breadth of the base should be about twice the distance from the pin to masonry; if made much larger, too much pressure will be concentrated on the area directly under the pin; if made much smaller, there is danger of overturning, although, of course, a great deal depends on the structural arrangement for carrying these stresses. A base plate more than 18″ in largest dimensions should be not less than $\frac{3}{4}$″ thick and no plate should project more than 4″ from a support and these supports should be not more than 10″ apart. When the ribs are not sufficient for this purpose, gussets, Fig. 61*n*, may be built out from them or partitions commonly called " diaframs " or " diaphragms," Fig. 61*m*, placed between them. Both

serve the important purpose of stiffening the ribs. When unsupported, the thickness should be at least one-twelfth the length.

The pin is of medium steel and should fasten both shoe and pedestal together to prevent a possible uplift, although the pin is often set in two half holes.

The shoe and pedestal may be cast or riveted. In the former case it may be cast iron or of cast steel, the latter being better but more expensive. The usual thickness of metal is $1\frac{1}{4}$ or $1\frac{1}{2}''$. Four bolts $1\frac{1}{4}''$ diameter are common for anchor bolts.

Riveted shoes can be made lighter than cast shoes and are less likely to injury from impact. Both shoe and pedestal

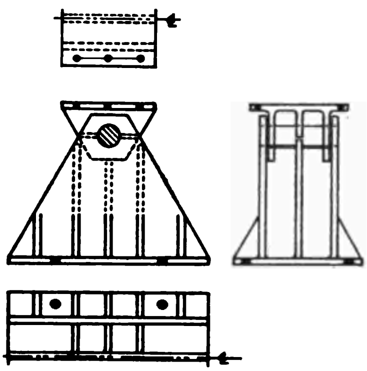


FIG. 61*o*.—Cast-Shoe of Type (*4*).

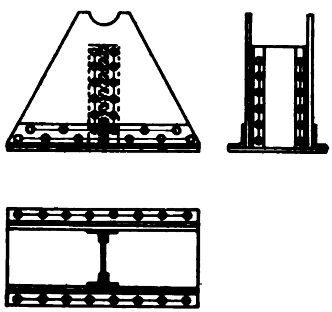


FIG. 61*p*.—Riveted Pedestal of Type (*4*).

are composed of horizontal plates to which ribs parallel to plane of girder or truss are attached by means of angles. These ribs are made of one heavy plate or 2 or 3 lighter ones riveted together and are planed where they bear.

Diaframs are made of 4 angles and a plate, Fig. 61*m*, while gussets are composed of 2 pairs of angles and a triangular plate, Fig. 61*n*.

Rivets in the base plate of the pedestal should be countersunk. Hence, as they have scarcely any stress, use maximum allowable spacing. The locking together is accomplished and at the same time a minimum of stress put in the pin by passing it through the outside plates of the pedestal and the inside plates

of the shoe as shown in Fig. 61*l*. A clearance of about $\frac{1}{8}''$ should be provided between the plates.

Type (*5*) Free with pin, rolling joint.

This type is made like type (*4*) except the roller nests and plates on which it bears, which are like type (*3*).

FIG. 61*q*.—Typical Shoe, Type (*4*). Showing Two Shoes Connected by an End Strut, American Bridge Co., Ambridge, Pa.

Art. 62. Structural Drawings

The organization of a structural drawing room is about as follows:

(*1*) Chief Engineer, in general charge of all engineering work, and particularly interested in questions of design and securing new contracts.

(*2*) Head Draftsman looks out for the minor questions of design, but his duties are largely executive, allotting work, handling correspondence, hiring and discharging men. He should be capable of encouraging a spirit of loyalty in every subordinate.

(*3*) Squad Bosses are placed in charge of a group of four to twelve men, and report to the head draftsman. They are expected to do actual work besides superintending that of the other men.

(4) Estimators, see Art. 53.

(5) Checkers, see Art. 66.

(6) Detailers make the shop drawings from strain sheets and write bills.

(7) Tracers copy drawings already worked out in pencil or write simple bills. They gradually take up detailing.

(8) Stenographers, blueprint boys, errand boys, and so on.

The drawing is preferably made on dull side of tracing cloth in order to show pencil marks clearly. An experienced detailer will make a drawing directly on it. Parts of simple structures may be inked in direct if no changes are likely. The common and about the best size is $24'' \times 36''$ outside with one-half inch border all the way around. Sheets about $10'' \times$

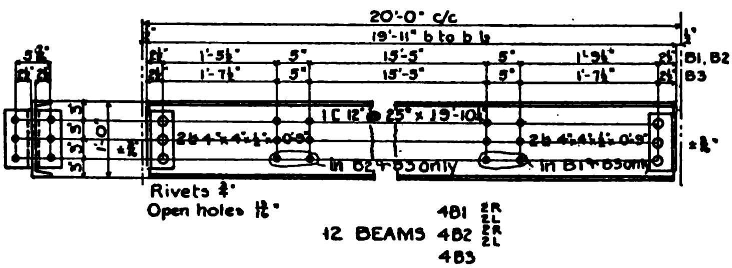


FIG. 62*a*.—Detail of Several Kinds of a Beam on One Sketch.

$12''$ may be used for detailing I beams, channels, and H sections. Often beam sketches with dimension lines are printed thereon.

Clearness and neatness are essential qualities of a structural drawing. Use $\frac{3}{4}'' = 1'$ for large or very simple work, and $1''$ for other ordinary cases. Special larger scales are employed only for complicated details or machining. Where work is of a uniform nature, a break may be inserted and only a portion of the length shown. Sometimes center lines of truss are made to one scale while details around each joint are much larger. Ignore scale where a clearer drawing may be secured. In changing dimensions, we seldom alter sketch. Do not attempt to crowd the drawing. Of course, every additional tracing means much more blueprinting. However, we believe that no money is gained by putting objects so close together that there is not the proper space between views and the different pieces. Always allow more room than you think you need,

because changes, errors, and lack of foresight tend to increase amount to be shown on a drawing.

In case several pieces are somewhat alike but not exactly so, they are detailed by one sketch, notes and separate sets of dimensions showing difference, Fig. 62*a*. Notice carefully and consult many other drawings because no set of rules, nothing but observation and practice, can make a draftsman.

Always follow a definite order. Fig. 62*b* shows method for various views. Omit those that are not necessary. If both ends are alike, omit one end view; if nearly so, show difference by notes. It is better if possible to place vertical members with their tops at the top of the sheet, and to draw horizontal members horizontally. It is wise in many cases to note "West" or "Mark this end 'Top'" for convenience in erection. In case structure is symmetrical about center line or nearly so, note it thus and detail left half. In constructing views, parts which would obscure or unnecessary things which would take too much time are often omitted. Do not shade except for curved surfaces, and in general avoid artistic or decorative work except perhaps in dealing with non-technical men in securing contracts.

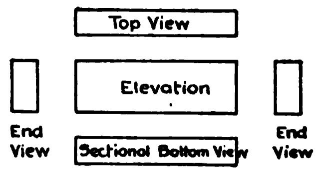


FIG. 62*b*.—Arrangement of Views.

Conventional signs are very little used except for rivets. Open holes, the rivets for which are driven in the field, should be drawn to scale and blackened. A shop rivet is drawn to scale of head and left open. Flattening the head is shown by small lines at an angle of 45°: inside the rivet if on inside or far side; outside if on outside or near side. The number of lines show the number of eighths of inches in height of rivet head; if countersunk and chipped an *x* is used thus:

Field rivet, full head . . . .
Shop rivet, full head . . . .
Field rivet, countersunk and chipped far side,
Shop rivet, flatten to $\frac{1}{8}''$ near side . . . .
Field rivet, flatten to $\frac{3}{8}''$ both sides . . . .

FIG. 62*c*.

A " marking diagram," a small line drawing of complete structure, is advisable in many instances. Members detailed on drawing are shown heavy therein. Small pieces which are bolted to larger ones for shipment must be so marked, Art. 48, (2).

The lines showing the object whether broken or full should be of medium weight. If a full even line is not secured, the ruling pen may be dull or the tracing may need a more thorough rubbing with pounce powder. On the other hand, dimension lines, center lines, and so forth, should be made as light as possible to ink readily.

Visible line, 1/80″ thick
Invisible line, 1/80″ thick
Dimension line, very thin
Center line, very thin
Section line, very thin

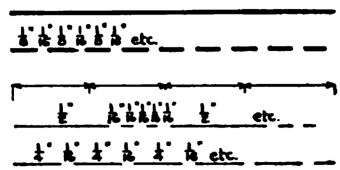


FIG. 62d.

Draw these out to indicated measurements on a piece of paper and then keep as near as possible thereto by eye. Uniformity and correctness of proportion are essential even in dotted lines.

The salient points of lettering are execution, form, spacing, and general arrangement. The one idea is uniformity. Better a lettering which is uniformly poor than one which is partly good and partly bad.

(*a*) The work done by a man in tracing an ideal bit of lettering may be termed his execution. Lines must not be blotted, blurred, or even ragged. For design work, they are made of medium weight. Here it is of utmost importance that width be uniform. Use a medium pointed pen which makes proper weight without pressure. Ruling pen must be employed altogether or not at all. Difficulties encountered by a novice are a lack of steadiness which practice will generally correct, and a blurring due to an unclean pen or a partially dried-up ink.

(*b*) Form. In every system of lettering, there are certain proportions which give best results. Beginners must master and use them. Furthermore, we must have uniformity in:

(*1*) Breadth of letter. Do not mix broad and narrow letters or figures. Except where crowded, make relative dimensions the same throughout the drawing and contract.

(*2*) Height. Make same class of letters same height for same job. For the bulk of the work, heights should be: for letters $\frac{3}{32}$ and $\frac{1}{8}''$; whole numbers, $\frac{7}{64}''$; fractions, $\frac{7}{32}''$. Some draftsmen do very well without guide lines, but most men need them. At any rate adjoining letters must have equal heights.

(*3*) Vertical or slanting letters may be used but inclination must be kept constant for same class of lettering and particularly in the same note or line of dimensions.

We give below, first correct lettering and then in turn errors (*b*), (*1*), (*2*), and (*3*).

Rivet Rivet Rivet Rivet *Rivet*

FIG. 62*e*.

(*c*) Spacing between letters of the same word and between different words should appear equal. This does not mean that they will be equal.

Rivet *not* R ivet

FIG. 62*f*.

(*d*) General arrangement should harmonize with the drawing. Place lettering near object to which it refers but avoid crowding as much as possible. For a long object use a long title and vice versa.

Notes should state material, size of rivets, and open holes, paint, and any unusual or important point in the specifications. They ought always to be placed at the same part of the drawing.

Title should give name of purchaser, location, span, and kind of structure, name of fabricator, scale, date, when made and by whom, when checked and by whom, and name of squad boss in charge.

Besides the drawings of structural steel, the detailer prepares those for the machinery, bills of various sorts, and the erection

diagram. The latter is a large line drawing for the use of the erector and checker. It gives principal dimensions of the structure, shows each part in its finished position, and states any directions which may be made necessary by any peculiarities of design. It should also have a list of drawings with number and contents.

Art. 63. Auxiliaries—Bills of Material

Auxiliary to the drawing are the following bills:

Material	Clevis Nuts
Eyebars, Plain	Bent List
Eyebars, Adjustable	Castings
Pins and Accessories	Field Rivets and Bolts
Pilot and Driving Nuts	Shipping Bill

These are written or lettered in ink on transparent paper so that they may be printed. At the top is the name of the company, its location, and so on, also a blank for name of purchaser, his location, date, number of sheet, name of draftsman, checker, and so forth.

The bill of material lists the steel required and combines it to make the mill order. If a logical procedure were followed, this would be written after details were made, checked, and accepted. Instead the order is commonly written as soon after contract is signed as a draftsman can be secured. When the drawings are accepted, enough material may be on hand to start job. At any rate, valuable time has been saved, but at an increased cost in drawing room. Rough layouts are made to determine length of sections and details at critical points. Due to the hurry and the rough nature of the work, errors are quite probable. Also purchaser's engineer may require changes that affect order. If material has not been shipped from mill, it may be changed. When done this way two bills are written; the first a rough one in pencil, the second a finished one in ink. The two mill orders must correspond except for changes.

The finished bill is as follows omitting heading,

Material.								Mill Order.			
Assembly Mark.	Number.	Description.	Section.	Length.	+	Remarks.	Computed Weight.	No.	Section.	Length.	Item No.
				4 Columns	$C1$	2R 2L Drg	1		Medium O.	H. Steel	
	4	Shaft Pls.	18"×½"	20'-9"	½"	Mill both ends		4	18"×½"	20'-9½"	169
	16	Shaft Ls.	6"×4"×½"	20'-9"	½	Mill both ends		16	6"×4×½"	20'-9½"	325
$c1$	8	Cap Ls.	4½"×3"×⅜"	1'-5"	....	cut from 5×3	....	2	5"×3"×⅜"	22'-9"	386
$d1$	4	Cap Pls.	14"×⅜"	1'-7"		Plane 1 side	....	1	14"×11/16"	6'-4"	182
$f1$	4	Conn. Pls.	8"×⅜"	1'-2"	....	Bent ∠	....	......	....	....	Stock
$m1$	24	Conn. Ls.	5"×3"×⅜"	1'-5"			....	......	Ordered above		386

At head of mill order, write kind of material. Next the pieces and mark as shown. Then list main members and afterwards details in some definite order.

If a shape be changed, for example, a $6'' \times 6''$ angle sheared down to $6'' \times 5\frac{1}{2}''$, note should be made under "Remarks," "Cut to $6'' \times 5\frac{1}{2}''$," while order column reads $6'' \times 6''$. If piece be bent, it is mentioned under the head of "Remarks." Such material should be ordered separately from the straight stuff because former alone is sent to the blacksmith shop. Also material bolted for shipment must be noted.

Sketch plates are those of irregular plan which are ordered exact size from the mill. When cost of rectangular plate is greater than cost of sketch plate at 10 cents per hundred pounds additional price plus value of scrap cut from rectangular plate, about $\frac{1}{2}$ cent per pound, sketch plates should be ordered. Also where plates are too large for shop shears. Its plan must then be placed under "Description," and "Mill Order" marked "Sketch Plate."

Pieces planed on ends should be ordered about $\frac{1}{4}''$ long for each mill; if on flat sides, add $\frac{1}{16}$ to $\frac{1}{8}''$ for each. For plain stiffeners, increase length by $\frac{1}{4}''$; if they contain crimps, add $\frac{1}{2}''$ for each plus its depth.

The length of bent angles for ordering should be computed on center of gravity line. Several inches are added to allow of manipulation in the forge shop.

I beams and channels are purchased in lengths called for on the drawings, allowance being made for a variation of $\frac{3}{8}''$ either way as already explained, Art. 58. Bars, plates, and angles, less than 15 feet long, are ordered in multiple, that is, to be cut from longer pieces. The most convenient lengths are about but not much over 30 feet. Keep number of items down by combining mill order for each sized section as much as possible. Do not order lengths exceeding limits published in hand-books without consulting mills. Try to keep mill order in even figures, never using fractions less than quarter inches. If necessary to have two rolled edges, plates must be marked, "U.M.," (universal mill, Art. 19); otherwise either U.M. or sheared plate may be furnished.

Latticing should be ordered as so many lineal feet. Allow for unused ends and waste in cutting.

Structural companies carry steel in the more common sizes and some accumulates from errors and alterations in plans. Unless specifications for material prevent, use all of this which will fit, and write " stock " under " Order No."

Art. 64. Bills of Eyebars, Pins, and Accessories

In the bill of eyebars just below the heading is placed a sketch of a short eyebar, Fig. 64.

In the above dimensions, W and T are given by the strain sheet, P_1 and P_2 are made $\frac{1}{32}''$ or $\frac{1}{50}''$ larger than pin which is computed as given in Art. 60. T_1 and T_2 are usually made the same as T, the exception being where the pin does not furnish sufficient bearing. This is usually safeguarded by the rule that it shall be at least $\frac{3}{4}W$. D_1 and D_2 are determined from the standard size of dies, tables of which are furnished the draftsman. These are so made that there is at least 30% excess metal in a section through the pin. See Art. 43.

The length L_1 is the center to center of joints. Then

$$L_1+\frac{P_1+P_2}{2}=L_2 \text{ and } L_1+\frac{D_1+D_2}{2} \text{ gives } L_3.$$

To obtain A_1 and A_2, compute gross area of eyebar outside of the width W running from center to center pin holes. Add 10% to allow for burning and waste and divide by W.

The bill for adjustable eyebars is similar to that of the usual fixed type except that at one end is an upset with thread for which length, diameter, and number of threads per inch must be given. One of the threads must be left handed, while the other is right. The turnbuckles or sleevenuts are specified by calling for certain standard sizes, for which adjustable ends must be fitted. The area left at the root of the thread must be at least 30% in excess of that in the body of the bar. The length must be such as to give the turnbuckle plenty of play. About 3" should be left between the ends of the eyebars. L_1, L_2, L_3, and A, are taken from upset end of bar.

Similarly in the bill of pins a sketch of pins and pin nuts and the necessary dimensions determined as stated in Art. 46

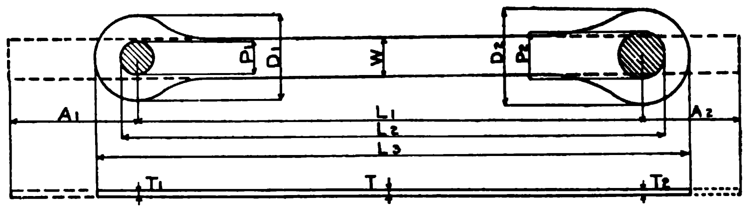


FIG. 64.

															Mill Order.			
Mark.	Number.	Width W.	T	P_1	D_1	T_1	A_1	P_2	D_2	T_2	A_2	L_1	L_2	L_3	No.	Section.	Length.	Item No.
L_0L_1	4	6"	1½"	5"+$\frac{1}{32}$	13½"	1½"	1'-9½"	6"+$\frac{1}{32}$	14½"	1½"	2-3"	20'-0"	20'-5$\frac{17}{32}$"	21'-2"	4	6"×1½"	24'-0½"	206
L_1L_2	8	6"	1"	6"+$\frac{1}{32}$	14½"	1	2'-3"	6"+$\frac{1}{32}$	14½"	1	2'-3	20'-0"	20'-6$\frac{1}{32}$"	21'-2½"	8	6"×1"	24'-6"	205

are given. As in the bill for eyebars, material is ordered on same sheet.

For pilot and driving nuts, it is simply necessary to call for the kind and give size of pin.

Art. 65. Other Bills

While a clevis nut may be billed in a manner similar to eyebars, it seems an unnecessary amount of trouble since they are usually one of several standard sizes, the only variation being in the size of thread, and opening of jaw. As in turnbuckles and sleevenuts, they must be threaded right and left on the same member. The bent list is a rough description of the work to be done by the blacksmith shop.

Castings may be placed on sheets for purposes of filing, or they may be detailed there without placing upon drawings at all.

The shop takes care of shop rivets and bolts. Those which are driven in the field must be listed, however. This is done on a bill which has columns for number, location, diameter, grip, and length under head. The latter exceeds the grip (thickness of metal grasped by rivet) by an amount sufficient to fill the hole plus enough to make the head.

At the end all rivets of the same length u.h. (under head) are summed up, and 10% added, to allow for errors and lost or condemned rivets. If in a place where extra rivets can be readily obtained, this bill need not be checked; if otherwise, a larger percentage may be added and the list carefully reviewed For erection in inaccessible locations, add liberally to longer lengths of rivets; in an emergency, a few might be cut down with cold chisel.

The shipping bill contains the marks of all parts and material to be shipped. The bulk of this is known simply by its marks. Often a rough description of each piece is added together with a statement of any pieces which may be bolted for shipment.

Art. 66. Checking

After the drawings and their accompanying bills have been completed, the checker examines them thoroughly and has the draftsman correct the errors. He places a small red check-mark or dot just over each correct word or dimension. Any which are wrong, he encircles in ordinary or blue pencil, writing correct amount. The author prefers to cross mark the dimension, perhaps explaining the reason to the draftsman, retaining the correct figure in his own note-book. This ensures proper re-examination by the detailer, which is often slighted. Furthermore an ignorant man is likely to erase or even change figures in correcting the drawing. Some companies keep "field checkers," who examine field holes and field clearances. We believe that it is unnecessary if precautions stated in this article are followed.

Some men check very roughly, running over the dimensions and examining a few points here and there. Others give attention largely to small matters which would not amount to much anyway. They are very fond of changing dimensions by a thirty-second of an inch; as a matter of fact, in their zeal for extreme accuracy, much more important matters may escape them. There is a great deal of friction between detailer and checker. Actual errors must, of course, be corrected, but the trouble arises over the design of the details. To avoid this:

(*1*) Let both detailer and checker consider the best good of their employer.

(*2*) If checker cannot convince a reasonable detailer of the wisdom of a change, it is better to let it go as it is. If the latter feels that his work will not be materially altered, he will take much more interest.

Next let us consider the good checker. He seldom finds it necessary to change dimensions by a thirty-second of an inch. In conference with the draftsman, he discusses minor errors of design and suggests means for avoiding them in the future. They consider together the more serious mistakes and the best method of correcting them. When cost of making change in drawing room equals or exceeds amount to be saved, he leaves

it alone unless deficient in strength. He is quite insistent on vital matters. No weak points, no infringement of specifications, no impracticable shop or field work escapes him. Especial care is taken with open holes and field clearances.

The following system may be used to great advantage. It is better to go over the entire drawing or set of drawings considering only one point at a time. Before finally signing the drawing, be sure that every word and dimension is check marked.

Schedule for Checking

(*1*) Check Principal Dimensions. Read correspondence to be sure no changes have been made by letter. Then check principal dimensions to agree with purchaser's drawing and also any interdependent sheets of details already checked. Above all, compare carefully with masonry plan. Want of agreement with this is certain to cause trouble. Be sure to examine arrowheads at the same time. The latter applies to *all* dimensions.

(*2*) Check Agreement with Specifications. If sizes were given in strain sheets or general plans forming a part of contract, they should now be compared. Otherwise, they must be designed from stresses or loads. In the same way, each joint, splice, connection, and rivet spacing in built-up girders, must be compared or thoroughly tested. Read specifications and see that every clause governing the work is satisfied. Particularly:

(*a*) Are ends of columns milled?

(*b*) Does thickness of material lie between minimum allowable and maximum practical? (Arts. 50 *d2*, 44.)

(*c*) Rivet spacing must not exceed nor be less than certain limits. (Fig. 47*g*, Art. 50*e1*.)

(*d*) Edge distances should lie within permissible values. (Art. 50*e1*.)

(*3*) Views and Notes.

(*a*) Are views properly shown?

(*b*) Are rights and lefts given where necessary and are they correct where stated? (Art. 40.)

(*c*) If detailed as symmetrical about center line, is it so mentioned and is it true?

(*d*) Is each necessary mark given, no two being alike?

(*e*) Are pieces to be bolted for shipment so designated? (Art. 48.)

(*f*) Do the remaining notes convey the correct information concisely?

(*g*) Are any other notes necessary?

(*h*) Check title.

(4) Shop rivets and bolts.

(*a*) Is templet work economized? (Art. 40.)

(*b*) Is spacing suitable for rack work? (Art. 44.)

(*c*) Can rivets and bolts be easily driven? Are rivets staggered where advisable? (Art. 47.)

(5) Miscellaneous.

(*a*) Have you good shop clearances?

(*b*) Is all material listed?

(*c*) Are there measurements for inspectors?

(*d*) Are enough dimensions given?

(*e*) Can lengths as detailed be secured from mill?

(6) Erection.

(*a*) Are pieces such as to admit of economical shipment? (Art. 48.)

(*b*) Consider some easy method by which structure as detailed could be put together. (Art. 49.)

(*c*) Check every piece to conform to this method.

(*d*) Test clearances during and after erection. Do this on each connecting piece, thus reviewing it at least twice.

(*e*) Can it be painted after erection?

(7) General.

We may now proceed to check lines of dimensions and see that they add up. Field holes should receive special attention and, like field clearances, be verified every place where they occur. For each group check,

(*a*) Vertical position and spacing.

(*b*) Horizontal position and spacing.

(*c*) Arrangement, how staggered.

(*d*) Diameter of holes.

(*8*) Bills.

Bill of rivets need not be checked when located where same may be readily purchased. Omit also the reviewing of shipping bills for jobs in the vicinity. Other bills must be checked. Use above outline where it will apply but in general these bills are simply listing. Ensure that everything is taken off by following your own procedure.

Above method is rather slow but in competent hands will obviate field checking and lessen bill for field extras. The latter is a statement of extra cost during erection caused by mistakes in drawing room and shop.

Art. 67. Other Steps

After the drawing is signed by the checker, the squad boss takes it to the head draftsman or chief engineer of the structural company, who examines the drawing, usually confining his criticism to matters of strength and shopwork. After his approval, the drawings are sent, usually all or a large part of a contract at a time, to the engineer of the purchaser. His examination is particularly for strength. If his concern does the erecting, he should investigate field connections and clearances; for, although the specifications frequently contain clauses making the seller responsible for any errors, it is better for both parties that they should be discovered in time. If the job is for a lump sum, he is careful that details are not skimped; if for a pound price, he guards against an excess of material. Changes ordered by either engineer are promptly made. Extreme care must be taken to see that everything affected is correctly altered.

As soon as both signatures are attached, blueprints are taken of all the drawings and bills. As each part of the works will need a print, from 10 to 20 copies of each must be taken. Any alteration, whether caused by error or by a change in design, must be corrected on the tracing and date noted in the title. Either all the prints which have been sent out must be called in and destroyed and a new set made from the revised tracing, or else " change slips " must be sent out to be pinned on the drawing. These give the alteration to be made and date and

name of checker. They are regarded in much the same light as notices from the registrar's office at a university.

Art. 68. Examination of Structures in Use

When called upon to examine a structure, the engineer should obtain the following data:

(*1*) General dimensions.
(*2*) Cross sections of members.
(*3*) Details of these members.
(*4*) Connections at joints.
(*5*) Accessories, such as flooring, ballast, etc.

Preceding information may sometimes be obtained from drawings. Very great care must be taken not to use proposed plans and even working details are likely to have been modified during or after construction. Often no trustworthy data can be found and complete measurements must be had if possible.

(*6*) Character of material.
(*7*) Deterioration through rot, rust, or other causes.
(*8*) Wear caused by traffic.
(*9*) Workmanship.

Number (*6*) may be found on plans, or material taken from a part of the bridge may be placed in a testing machine. The latter is the approved method of determining injury caused by fire or an accident of some kind. (*7*) is obtained for steel by scratching off rust and comparing former and present thickness. The condition of timber may be well tested by boring a hole in a position where it will least weaken the piece. (*8*) may be very easy to see as in the planking of a highway bridge or very difficult as in the shank of rivets. In general the wear of structural work or its overload is hard to detect by visual examination. Loosened rivets and excessive deflection are common signs. Stresses near or beyond the elastic limit result in those manifestations familiar to all—necking down in tension and bending in compression. And finally:

(*10*) Loads to which the structure will be subjected.

Taking now a standard set of specifications, compute permissible stress in weakest part of each member. Allowance must be made for deterioration, wear, or poor workmanship,

if such be present. Now compare this with actual total stress. If excess of latter over former be less than 10%, it is all right; 10 to 20%, it needs close supervision; 20 to 40%, is dangerous; above 40%, must be replaced at once. Of course, these are general rules to be used with judgment. Carefully consider such points as efficiency of supervision, loss of life, limb or property, in case of accident, and probability of occurrence of maximum load.

Art. 69. Failures

Space will not permit us to discuss all prominent structural failures. We shall take up only a few which will be instructive. One purpose is to give the young designer a proper appreciation of the responsibility which he bears.

We think the record is not a bad one when considered as a proportion. Furthermore, the continual changes in the art of bridge building have deprived us of the light of experience. Pressure for extreme economy in engineering, material, and construction have made very narrow margin for the designer. What wonder if the bounds are overstepped at times? Such was the cause of the disaster to the Quebec bridge, the most important of recent times.*

This structure, Fig. 69*a*, was a cantilever span across the St. Lawrence River, with two anchor arms of 500 feet, two cantilever arms of 562.5 feet, and one suspended span of 675 feet. The latter two made distance between piers 1800 feet, 90 feet longer than the Forth Bridge, the longest existing span. The Quebec Bridge was 67 feet center to center of trusses and had a maximum depth of 315 feet. It was intended for two railway tracks, two electric car tracks, two roadways, and two footways. While building out as a cantilever beam, Aug. 29, 1907, it fell into the river, 150 feet below, Fig. 69*b*. Seventy-four lives and two million dollars were lost.

The causes were: Assumed dead load was only about 75% of the actual, too high stresses were allowed, and, by far the most important, insufficient latticing was provided for compression members. The lattice bars of a bottom chord member,

* See Engineering News, September 5, 1907, et seq.

FIG. 69*a*.—Quebec Cantilever Bridge before its Fall.

FIG. 69*b*.—Wreck of the Quebec Bridge.

Fig. 56*s*, broke when its stress was 14,000,000 lbs., or 17,900 lbs. per sq.in. This and other columns in the vicinity suffering about the same unit stress had been showing signs of overload. Tests on a model post, one-third size, showed an ultimate compressive strength of 22,000 lbs. per sq.in. A better latticed but somewhat similar column bent in body at 30,000 lbs. per sq.in. In other words, the compression strength of large pieces of good design is not far from *25,000 pounds per square inch.*

We may divide failures into two classes, those which result from:

(*1*) Some fault of design or maintenance, and

(*2*) A very unusual occurrence. Such are earthquakes, extreme storms, or floods, derailment or collision on a bridge. Provision can be made for these accidents and sometimes this is done. However, generally speaking, one is not justified in spending money for remote contingencies.

We may further subdivide (*1*):

(*1a*) Ignorance. Such was the case in the disaster which occurred on the Boston and Albany Railroad at Chester, Mass., in 1893.* Here were located two through skew spans, each of riveted quadruple latticed trusses. Workmen were strengthening the bridge by placing additional cover plates on top chord, Fig. 69*c*. To do this, old rivets were driven out, replaced by bolts, the plates added, and rivets substituted for bolts, a few at a time. The foreman does not seem to have fully appreciated the function of the rivets and allowed too many holes to remain empty. A train of an engine and eight passenger cars was passing over the thus weakened structure when it broke, killing 17 and injuring 32.

FIG. 69*c*.—Section of Top Chord of Chester Bridge.

(*1b*) Economy in two forms:

(*1b1*) Saving in first cost so extreme that failure results as in the Quebec bridge.

(*1b2*) Keeping a structure in service under loads much heavier than those for which it was designed. The writer once visited a wreck caused by a mistake of this sort. As near as he can recollect, it was a railway deck plate girder of 65 feet span.

* Engineering News, September 7, 1893, et seq.

Web was 5 feet deep, made up of $36'' \times \frac{5}{16}'' \times 5'-0''$ plates, spliced by two bars and a single row of rivets on each side. Flanges were each of 2 Ls, $6'' \times 6'' \times \frac{5}{16}''$ and 2 cover plates $14'' \times \frac{5}{16}''$. Stiffeners were 6 to 9 feet apart.

(*1c*) Lapses.

We shall so term cases where an engineer, otherwise skillful, has shown incompetence or forgetfulness in one particular respect. As such we shall class the Tay Bridge disaster.* This occurred at the Firth of Tay in Scotland, Dec. 29, 1879. The structure was a viaduct about two miles long and contained 85 spans varying in length from 27 to 245 feet. The 13 that fell were through riveted lattice trusses of 245 feet span. They were supported on hexagonal towers whose legs were of cast iron. On the night in question, a storm was raging and the wind was blowing against the truss with a speed estimated at 72 miles per hour. When the engine with seven coaches had almost reached the middle of the thirteen spans in question, the whole blew over. Not one of the 75 persons on board survived. The structure, otherwise well designed, lacked strength to resist wind stresses. Failure was probably due to weakness of laterals and their connections by lugs to the cast-iron columns.

(*1d*) Unforeseen conditions.

We refer here, not to the extremely improbable conditions which we have already discussed but rather to those which are probable and should be guarded against.

Such a case occurred in the Horne building fire in Pittsburgh in May, 1897.† A large part of the damage was caused by the fall of a water tank on the roof. This was carried by naked beams protected only by a suspended ceiling below. When fire occurred, it wrecked this ceiling, and the heated beams allowed tank to fall. Beams themselves should have been fireproofed.

We will give in addition two more historic failures.

The Ashtabula accident happened on the night of Dec. 29, 1876.‡ The bridge was a deck, double track, Howe truss built of iron. The trusses had 14 panels at 11 feet, were $19'-9''$

* Engineering News, January 3, 1880.

† Engineering Record, Vol. XXXV, p. 537.

‡ Engineering News, January 6, 1877 et seq.

high, located 17′–2″ c. to c. Compression members were each made of several small I beams with practically nothing to correspond to modern latticing, and details were very poor. As a heavy train with two locomotives and eleven cars was passing slowly over the bridge, it collapsed just as first engine had nearly reached the other abutment. The train fell 75 feet and was consumed by fire. Of 209 persons on board, 92 were killed and 64 injured. It is supposed that failure occurred in the top chord under the first locomotive.

The Bussey Bridge near Boston, Mass., was a skew bridge of 104 feet span.* One truss was a deck Whipple of 16 panels at 6.5 feet and depth 12.5 feet. Other was 4 panels at 26 feet and depth 16 feet. Pin through floorbeam was attached to pin at hip joint by an eccentric loop welded hanger made of two bars about $1\frac{3}{8}'' \times 1\frac{7}{8}''$. This was improperly designed and broke, allowing latter part of a long train to fall through the bridge; 32 were killed and 70 hurt.

We shall not attempt to enumerate the troubles of standpipes and highway bridges. Failures of the former are frequent and not always easy to explain. The weakness of latter is caused by:

(*a*) The election of politicians rather than business men or engineers to committees having the matter in charge.

(*b*) The tendency of the layman to rely rather upon the advice of salesmen than upon that of reputable designers.

(*c*) Attempts to economize. Except when aided by experts, this is very poor policy. Only a trained man can tell whether a low price represents inferior work or not. Usually it does.

(*d*) The economy which precludes expert advice, operates in inspection of details and field erection.

By far the greater portion of accidents that have come to the author's attention have been due to slowly applied or quiescent forces. Some of the structures mentioned above had at times carried rapidly moving loads. However, the failure did not take place then but later with less impact.

* Engineering News, March 19, 1887 et seq.

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